CHARACTERIZING AND TESTING OF MODIFIED OIL WELL CEMENTS, PIPE JOINTS AND MANHOLE COATING

A Thesis

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In Partial Fulfillment

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Master of Science

in Civil Engineering

by

Saravanan Ravichandran

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ABSTRACT

Some of the challenges in maintaining the onshore and offshore infrastructures are in monitoring the performance of cementitious materials used in construction and the performance of joints in the storm water pipelines and coating on manholes. In order to monitor the setting time of oil well cement used in cementing oil wells and Portland cement used in many other construction were modified with additives to make the material more sensing. The curing of the modified oil well cement was compared with Portland cement by monitoring the changes in electrical resistivity. The curing behavior of modified oil well cement and Portland cement with and without foam showed the changes in resistivity with the curing. Modified oil well cement and Portland cement also exhibited enhanced piezo-resistive sensing property under compression load and hence can be used for monitoring the material's behavior.

Multi wall polypropylene pipes are increasingly used in storm water applications. Based on a test protocol developed, polypropylene storm water pipe joints were tested under various loading conditions. Finite element model was used to investigate the pipe at the joints and the deflections were compared to the experimental results. Concrete and claybrick manholes were coated and tested under increasing hydrostatic pressure up to 11 psi for 21 days. The coating movement was measured during the testing period.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

The storm water and waste water infrastructure assets in the USA amount to several trillions of dollars. These infrastructures not only ensure the quality of life in the community but also the growth of the commercial activities. These infrastructures include thousands of miles of pipelines and millions of manholes. The long-term performance of the pipe lines are very much influenced by the performance of joints in the pipelines. Infiltration in the joints of the storm water and waste water pipelines are causing many maintenance problems. Hence it is important to ensure that the new pipes used in storm water and wastewater applications have quality joints to meet the design requirements. Also, corrosive environment in the manholes are affecting the long-term durability of the concrete and brick manholes. Hence new coating materials are being developed to protect the manholes. The applicability of these coating materials must be evaluated under service conditions. Various types of cements are used in the construction of onshore and offshore structures. There is increasing interest in improving not only the performance but also the sensing properties of cementitious materials used in installations and maintaining the off-shore and on-shore infrastructures.

The Gulf of Mexico oil spill (also referred to as the BP oil spill) was the largest accidental marine oil spill in history. The oil rig was operated by British Petroleum (BP). It claimed 11 lives following the explosion of the oil rig. A sea-floor oil gusher flowed unabated for 87 days. The estimated total discharge was 4.9 million barrels. Investigations confirmed that the main cause of this accident was due to the defective cement in the oil well. Hence it is essential to ensure the quality of both casing joints and the outer oil well cement. In this report, testing methods of oil well cement and pipe joints were broadly discussed.

1.1 Oil Well and Portland Cements

In oil and gas wells, cement slurries are used for cementing the steel casing to the wellbore. Also it is used for sealing the rock formations from the well (Becke et al., 1998). The casing consists of layers of huge pipes with joints in between. If the joint leaks, it will release the harm gases into the outer oil well cement and adversely affect the oil well cement surrounding it. Hence it is essential to ensure the quality of joints in the drilling rig.

Cements such as class G and class H are used in oil well cementing applications. These cements are produced by pulverising clinker consisting essentially of calcium silicates with the addition of calcium sulphate (John, 1992). Cementing is an important operation at the time of oil well construction (Backe et al., 1997). When admixtures are added with cement, tensile and flexural properties will be modified. Also admixtures will have effect on corrosion resistance, drying shrinkage, thermal conductivity, specific heat, electrical conductivity, and good radio-wave reflecting and absorbing properties of oil well cement (Bao-guo, 2008). Oil well cement slurry will be used several thousand feet below the ground level. Hence determining cement setting time is always a challenge.

The quality of oil well cement is important as it will directly impact on various operations during drilling (Backe et al., 1997). If the quality of cement is bad, it will add unnecessary cost to the drilling operation. But the high quality of cement will provide a high quality casing which in turn will have the long-term durability of borehole (Pourafshary et al., 2009). Setting time of oil well cement is very important to control the cementing operation (Zhang et al., 2010). It will be difficult to pump, if the cement sets too soon. Hence it is important that this limit not be reached before the cement slurry fills the annulus (Smith, 1990). Water-to-cement ratio (w/c), the composition and particle size distribution of cement, presence of additives (mineral and chemical), temperature, and pressure may affect the setting time of cement (Taylor, 1997). The cement slurry will become a rigid material from a workable plastic paste after setting. If the amount of product becomes sufficient to cause the particles to overlap, setting will begin. It is referred to as the percolation process (Zhang et al., 2010).

1.2 Testing of Pipe Joints

The long-term performance of buried storm water and waste water infrastructure are very much affected by the condition and quality of the pipe joints. Groundwater infiltration or rainwater infiltration and inflow (I/I) through pipe joints are basis for sewer rehabilitation (Lee et al., 2009; Wirahadikusumah et al., 1998), which are considered problematic. The defects in saturated soil are common and ground water finds its way through these defects to enter into combined sanitary and storm sewer systems. However, sanitary and storm sewers are mostly affected by infiltration and rainwater inflow.

Rain-induced infiltration also gets into the storm sewers, by many ways such as defects, false plumbing connections or openings in manhole covers (Staufer et al. 2012). This rain-induced infiltration will add extra load to the underground drainage pipes. As a result sewer water treatment plants will get affected by extra load of waste water as well. This will increase the cost of the treatment substantially (Water Environmental Federation, 1999). Also the leaking water from the damaged systems will erode the soils around the drainage pipes which will result in the ground surface settlement. Due to the ground surface settlement, drainage pipes above will get moved, which will damage the pipe joints considerably. There are few test methods to assess the performance of drainage pipe sizes varied from 3 to 144 inches. Infiltration/exfiltration tests use both air and water as recommended. From the records reviewed, most of the tests were of exfiltration (85%) and only less number of tests (23%) was of infiltration tests. Also, test methods for shear force tests at the joints and misalignment (angular) tests are recommended for plastic, fiberglass and concrete pipes. From 3.5 to 40 psi pressure are used for performing the tests. Based on type of pipe and application, acceptable leak rates are varied (Vipulanandan et al., 2005).

1.3 Rehabilitation of Manholes

Concrete manholes are used not only to access but also to allow for aeration of drain or sewer systems conveying sewage or surface water under gravity. During rainy season, the hydrostatic pressure on the manhole will increase because of raising water table. The concrete manhole must withstand such high water pressure without failure. Due to corrosion and erosion of the manhole internal walls coating materials are used to protect surfaces. When coatings are used to rehabilitate manholes, it will be subjected to the external hydrostatic pressure. Hence the performance of the coatings must be evaluated using full-scale tests. The bonding strength between concrete and coating must be high enough to prevent water infiltration into the manhole. So Bonding between the concrete surface and the coating material is another important property that must be evaluated to determine the performance of the coating.

Bricks have been used as a construction material throughout the world for over 5000 years. Still bricks are serving for the same purpose (Karaman et al., 2006). During rainy season, the brick manhole should be strong enough to withhold the high hydrostatic pressure due to the high water table. Coatings will be subjected to the external hydrostatic pressure as it is used to rehabilitate manholes and other infrastructures. Hence the performance of the coatings must be evaluated using full-scale tests.

1.4 Objectives

The overall objectives were to evaluate some of the critical issues related to the behavior of modified cementitious materials and maintenance of storm water infrastructures. The specific objectives of this study are as follows:

1) Characterize the modified oil well cements based on the sensing properties during setting and after hardening.
2) Evaluate the joints of polypropylene pipes used in storm water infrastructure.

3) Test a coating for manhole rehabilitation.

4) The experimental study included testing of material and full-scale testing of the infrastructure components.

1.5 Organization

This thesis is organized into eight chapters. Chapter 1 is the introduction with the specific objectives for this study. Chapter 2 summarizes the background and literature review related to characterization of oil well cement, polypropylene pipe joints and manholes. Particularly, studies related to self-sensing ability of oil well cement during and after setting is reviewed. Chapter 3 discusses about both full scale and laboratory test of manholes, polypropylene pipe and oil well cement. Especially hydrostatic pressure test on manhole coating, plastic pipe and setting time test on oil well cement. In Chapter 4, properties of oil well cement and effect of admixtures on oil well cement with respect to the resistance measurement during and after setting. Performance of polypropylene pipe joints is discussed in chapter 5. In Chapter 6, experimental testing results, application and performance of manholes are presented. Finite element modeling of polypropylene pipe performance is included in chapter 7. Finally, the conclusions and recommendations of this research have been summarized in Chapter 8.

CHAPTER 2

BACKGROUND AND LITERATURE REVIEW

This chapter provides a review of the information in the literature related to this research. Studies on application of coatings to concrete and brick manholes and past, present studies on oil well cements, plastic pipe joints are also discussed.

2.1 Oil Well Cements

2.1.1 Introduction

Oil well cement is hydraulic cement. Under the high temperatures, it has a slow setting rate and it is used to support oil well casing.

Class G cement is produced from a raw meal containing an argillaceous component (clay or shale), a calcareous component (such as chalk or limestone), a source or iron oxide (such as haematite or pyrites residues). A small addition of quartz sand to allow sufficient silica to be present in toto in the raw meal, if necessary. The raw meal composition is designed to produce a clinker of suitable reactivity for oil well cement usage (John, 1992).

Gypsum addition is normally kept low, giving total cement SO_3 content within the range 1.7-2.3% (again to minimize the acceleration of the hydration reaction of tricalcium silicate (alite) with sulphate). Higher SO_3 levels may be tolerated if the total alkali content is low (John, 1992). Class H cement is produced by a similar process, except that the clinker and gypsum are ground relatively coarser than for a Class G cement, to give a cement with a surface area generally in the range 220-300 m^2/kg (John, 1992).

2.1.2 Temperature and Pressure

Class G and H oil well cements are considered to be basic well cement. Class G and H cements are suitable to be used up to 8000 ft depth under the ground. Also it can be modified with different accelerators and retardars in order to be able to cover a wide range of well depths and temperatures. Oil wells have variety of temperature and pressure profiles which are listed in Table 2-1.

Table 2-1: Temperature and pressure profiles of Oil wells (Smith, 1990)

Depth, ft	**Circulating Temperature, F**	**Static Temperature, F**	**Pressure, psi**
1000	80	92	700
6500	120	158	3850
9800	150	198	6160
14300	200	252	9655
18300	250	300	13285
21750	300	341	16650

2.1.3 Chemical Composition

According to the American Petroleum Institute (API) standards, Class H, G and Portland cement are used for this study. Chemical components of class H, G and Portland cement are listed in Table 2-2.

Table 2-2: Composition of Cements (Zhang, 2010, Mindness and Young, 1981, Zhou,1996 & John, 1992)

	C_3S	C_2S	C_3A	C_4AF	Gypsum	Surface area (m^2/kg)
Class H	63.94	15.84	0.57	11.33	1.8	220-300
Class G	56.5	18.06	1.17	14.29	--	270-350
Portland Cement	50	25	12	8	3.5	----

2.1.4 Chemical Reaction During Hydration

When the cement is mixed with water, the binding phases in Portland cement (Ca_3SiO_5, Ca_2SiO_4, $Ca_3Al_2O_4$, and $Ca_4Al_nFe_{2-n}O_7$) react in different ways. During hydration of Portland -

cement, chemical reactions between the clinker components, calcium sulfate and water would take place, which will lead to continuous cement slurry thickening and hardening. In the Portland cement, more than 80% of the total material would be of silicate phase. Calcium silicate hydrate and calcium hydroxide are the products of hydration of both tricalcium silicate (Ca_3SiO_5) and dicalcium silicate (Ca_2SiO_4) which is shown below (Natarajan, 2005),

$$2\ Ca_3SiO_5 + 7\ H_2O \rightarrow 3\ CaO \cdot 2\ SiO_2 \cdot 4\ H_2O + 3\ Ca(OH)_2\ , \qquad (2\text{-}1)$$

$$2\ Ca_2SiO_4 + 4\ H_2O \rightarrow 3\ CaO \cdot 2\ SiO_2 \cdot 4\ H_2O + Ca(OH)_2. \qquad (2\text{-}2)$$

The resulting product 'Calcium Silicate Hydrate' gel is the principal binder of hardened cement. Also Calcium Silicate Hydrate comprises almost 70% of fully hydrated Portland cement. At short hydration times, $Ca_3Al_2O_4$ (aluminate phases) are the most reactive. Aluminate phases have a significant influence upon the rheology of the cement slurry and early strength development of the set cement. But their presence is relatively small compared to the silicates. Tricalcium aluminates ($Ca_3Al_2O_4$) hydration reaction are shown in the equation below (Natarajan, 2005),

$$2Ca_3Al_2O_4 + 27\ H_2O \rightarrow Ca_2 \cdot Al_2O_3 \cdot 8H_2O + Ca_4 \cdot Al_2O_3 \cdot 19H_2O. \qquad (2\text{-}3)$$

The calcium aluminate hydrates (C_2AH_8 and C_4AH_{19}) will be converted into more stable form $Ca_3 \cdot Al_2O_3 \cdot 6H_2O$ from its metastable form by the reaction as shown in the equation below (Natarajan, 2005),

$$Ca_2 \cdot Al_2O_3 \cdot 8H_2O + Ca_4 \cdot Al_2O_3 \cdot 19H_2O \rightarrow 2\ Ca_3 \cdot Al_2O_3 \cdot 6H_2O + 15H_2O. \qquad (2\text{-}4)$$

$Ca_3Al_2O_4$ hydration is controlled by addition of 3 to 5% gypsum to the cement clinker before grinding. Upon contact with water, part of the gypsum dissolves. The calcium and sulfate ions released in solution react with the aluminate and hydroxyl ions released by the $Ca_3Al_2O_4$ to form a calcium trisulfoaluminate hydrate, known as mineral ettringite (Natarajan, 2005).

2.1.5 Additives

Studies were done by various researchers on the effects of additives on oil well cement and Portland cement. Roshan (2010) found that addition of carboxymethylcellulose (CMC) polymer along with some other additives with the oil well cement improved early compressive strength and rheological properties. Also it reduced the permeability of oil well cement. As reported by Choolaei (2012), when SiO_2 nanoparticles were mixed with the cement mortars, the rheological properties of cement slurry was improved, producing an increase in the compressive and flexural strengths of the cement mortar. Nano silica helped to eliminate the free water in the designed cement slurries. The setting time and the length of the dormant period were decreased with the addition of nanosilica with cement.

2.1.6 Effect on Setting Time

Ludwig (1951) reported that addition of sodium chloride in the oil well cement markedly changed thickening time, strength and physical properties. It was concluded that salt content in the range of 0 to 100,000 ppm decreased the thickening time and increased the early strengths and salt content range of 200,000 to 300,000 ppm increased the thickening time back to its normal while reduced the early strength.

O'Neal (1954) investigated that the setting time of cement slurries will get shorten by 48% under a pressure of 5000 psi while it is shorten by 63% under a pressure of 8000 psi.

Ridha (2010) evaluated the influence of water to cement ratio and degree of hydration to the dispersion characteristic of oilwell cement slurries during 24 hours of hydration. It is observed that the magnitude of conductivity dispersion increased with the reduction in water content and pore volume. At initial hydration, the dispersion characteristic appeared in the minimum value and rose as hydration continued. The frequency effect was growth as the hydration and saline concentration increased. The results gave an implicit correlation to the interface conductivity phenomena as indicated from dispersion respond through hydration process.

Zhang (2010) the relations among these methods were discussed. Different dosages of the superplasticizers were used to obtain cement pastes with yield stress <6 Pa at 30 min at given water-to-cement ratios. The results showed that the pastes with PLS lost workability more slowly and had longer initial setting time compared with those with PCE and PNS admixtures. Although the longer workable time is beneficial for hot weather concreting, the longer initial setting time of such material has to be taken into consideration where early strength development is essential.

2.1.7 Piezoresistive Behavior

Banthia (1994) observed that strength and durability of concrete was improved by the addition of small amounts of fiber-reinforcement. Due to the fiber's high resistance to wear, heat, and corrosion, carbon fiber-reinforced concrete in particular has been shown to have excellent durability properties.

Also, Chung (1996) reported that addition of carbon fiber to cement provided the strain-sensing ability and increased the tensile and flexural strengths, tensile ductility and flexural toughness, and decreased the drying shrinkage.

Chung (2001) studied that an increase of the measured electrical resistivity of carbon fiber-reinforced cement paste during resistivity measurement was induced by electric polarization. By increasing the conductivity of the cement paste through the use of carbon fibers that were more crystalline, the increase of the fiber content, or the use of silica fume instead of latex as an admixture, this effect can be diminished. The conductivity of the composite and the extent of polarization were increased by Intercalation of crystalline fibers. It was concluded that when the four-probe method was used, voltage polarity switching effects were dominated by the polarization of the sample itself, but when the two-probe method was used, voltage polarity switching effects were dominated by the polarization at the contact sample interface.

Reza (2003) proved that with the addition of a small volume of carbon fibers into a concrete mixture produced a strong and durable concrete and made the product as a smart material. It is recommended that these techniques could be used as nondestructive testing methods to assess the integrity of the composite.

Table 2-3: Literature review on Piezo-resistive study

Reference	Materia-ls	Modificat-ion	Tests	Resistivity change	Result	Remarks
Chung et al. (2008)	Cement, silica fume	Fiber of size 5mm	Compressive, tensile and flexural strength	--	Ultrasonic waves effect on improving the dispersion	Compressive, tensile strengths increased, flexural strength decreased
Reza et al. (2004)	Cement, silica fume, sand	0.6% of carbon fiber	Tensile strength test	--	Tensile strength increased by 3 times	Tensile strength increased
Vipulanandan & Sett (2004)	Polyester resin, sand	6% of Short carbon fiber	Tensile strength test	Less than 100 Ω.cm at 6% CF	Tensile properties changed	Tensile strength modified
Manuela & Raffaele (2004)	Cement, sand, silica fume	0.15% of carbon fiber	Setting time test	Conductivity 0.1 $(\Omega.cm)^{-1}$	Hydration time, percolation, curing	Curing effect on conductivity was studied
Chung et al. (1999)	Cement, latex, silica fume	0.5 & 1% carbon fiber	Thermo electric power measurement & see back coefficient	--	The linearity and reversibility of the see back effect is increased	The see back effect is increased
Chung et al. (1999)	Cement, silica fume	0.5% carbon fiber	Tensile strength test	3x10e-05 Ω.m	Tensile strength increased by 56%	Studied the effect of water reducing agent
Chung et al. (1999)	Cement & water	2.6 to 7.4% carbon fiber	Tensile strength test	5.91x10e-04 to 1.86x10e-04 Ω.m	Electric resistance increased upon tension	Tensile strength was sensed
Abo El-Enein et al. (1995)	Cement, sand and silica fume	-	-	-	electrical conductivity is influenced by addition of silica fume	Value of conductivity was increased to a peak at 1-3 hours of hydration and then decreased gradually
Remarks	Cement, silica are mostly used as base materials.	Carbon fiber of 0.125% to 7.4% is used.	Compressive, tensile and flexural tests are mainly done.	Resistivity is varied from 3x10e-05 to 100 Ω.cm	Mechanical properties of cement slurry are increased with addition of fibers.	

2.2 Pipe Joint Materials

Polypropylene $(C_3H_6)n$ is part of the polyolefins family. It is a versatile thermoplastic material used in many commercial applications (Hoppe, 2011). It is produced by polymerizing propylene in the presence of a catalyst. The starting material, propylene, is called the monomer, and the final compound is called the polymer (Hoppe, 2011).

The Virginia Department of Transportation evaluated the performance of 120 feet triple wall PP pipeline in three different routes and 80 feet of dual wall PP pipe in to different routes for storm water drainage (Hoppe, 2011). The diameter of pipe size was varied from 30 inch to 48 inch. The width of trench was 9 feet and depth of trench was 6 feet. The depth of burial was varied from 1.6 to 2.5 feet. The backfill type was crushed aggregates. From year 2009 to 2011, there was no crack found in the PP pipe and the maximum deflection was 4.5% which was acceptable as per ASTM standard. The joint separation was within 0.25 to 1 inch. In the UK, a twin wall and annular corrugated wall PP of diameter 22.78 to 25 inch was evaluated in the year 2000 (Faragher, 2000). The width of trench was 3.94 inch and the depth of trench was 5.25 inch. The type of back fill used was sand and gravel. The maximum deflection was within the acceptable range of 1.48 to 3.24 inch. This pipe was used for storm water drainage purposes.

The American Concrete Pipe Association (2005), tested a corrugated HDPE pipe of diameter 36 to 48 inch and length of 130 to 984 feet in Lexington, Kentucky. The pipes were installed from 1986 to 2000. The pipe deflection varied from 4 to 50% of pipe diameter which was not acceptable as per ASTM standard. Also cracks were found in the pipe sections.

In another study, (by the American Concrete Pipe Ass., 2000), a HDPE pipe of diameter 30 to 60 inch and length 15 to 70 feet was evaluated in 2002 for storm sewer applications. The pipe was buried at the depth of 2 to 5 feet and tested for 10 years. The maximum deflection of the pipe was 14.87% of pipe diameter which was not acceptable as per ASTM standard. The crack length in the pipe varied from 7 to 24 inch. The amount of joint separation was in the range of 1.5 to 5.5 inch.

A fiberglass pipe was evaluated in Riga, Latvia in 1983 (Howard, 1994). The diameter and length of pipe was 36.8 inch and 5.5 feet respectively. The size of the trench was 5.35 feet. The pipe was buried at the depth of 10 feet with the backfill material of loose sand and clay. The maximum deflection recorded was 12.7% of pipe diameter which was not acceptable.

Also, a corrugated PVC and HDPE, with the pipe diameter of 18 and 36 inch were evaluated in 1992 and were used in sanitary sewer applications. The backfill materials used for this pipe are ODOT#310 sandy and ODOT#57 stone and ASTM class III soil. The crack length formed was 3.125 inch. The maximum deflection recorded was above 5% of pipe diameter which was not safe and acceptable by standards.

The performance of PVC pipe was evaluated in 1990 in Elba, Nebraska (Howard, 1990). The size and length of the pipe was 27 inch and 20 feet respectively. The pipe was buried at 15 feet from the ground surface with poorly graded sand and clay. The size of the trench was in the range of 11 to 13 feet. The pipe installed year was 1987. The maximum deflection of pipe was 9.4% of pipe diameter which was again not accepted by ASTM standards.

Based on the above discussion, HDPE, PVC, Fiberglass and PP pipes have been evaluated across the world. The diameter of the various types of plastic pipes varied from 18 to 60 inches. Length of plastic pipes tested varied from 4.8 to 984 feet. Most of them were tested for use in sanitary sewer and storm sewer with backfill of sand, clay and stone. Joint separation of plastic pipes varied from 0.25 to 5.5 inch (Hoppe, 2011; American Concrete Pipe Ass., 2000). When it comes to the deflection, except for polypropylene pipe, for most of the other plastic pipes tested under various conditions the deflection exceeded 5% of pipe diameter which was not acceptable as per ASTM standards (Hoppe, 2011; Faragher, 2000).

Table 2-4: Case studies on polypropylene pipe

Reference	Type of pipe	Location	Pipe Size (inch)	Length of pipe (ft)	Trench size(Ft)		Type of backfill/ depth of burial(ft)	Application	Max % of deflection	Joint Separation (inch)	Remarks
					Depth	Width					
Hoppe (2011)	PP(Dual &Triple wall)	Virginia	30 to 48	30 to 55	6	9	No.26 Crusher run aggregate/1.6-2.5	Storm water drainage	1.6 to 4.5	0.25" to 1"	Max.deflection is less than 5% - acceptable
Coombs (2010)	Corrugated HDPE	Texas	48	4.8	-	-	#57 stone(ASTM class 1 soil type) & ASTM 2321 class III onsite soil	Storm water drainage	1.5	<0.25	Max.deflection is less than 5% - acceptable
American Concrete Pip Ass. (2005)	Corrugated HDPE	Lexington KY	36 to 48	130 to 983.6				Storm water drainage	<4 to 50	-	Max. deflection is 50% way higher than the ASTM standard of 5%
Faragher (2000)	PP (Twin wall)	UK	22.78 to 25	-	5.25	3.94	Sand & Gravel	Storm water drainage	1.48 to 3.24	-	Max.deflection is less than 5% - acceptable
American Concrete Pipe Ass. (2000)	HDPE	USA	30 to 60	15 to 70		-	None/2-5	Culvert drain(Storm) applications	5.83 to 14.84	1.5 to 5.5"	Max.deflection is greater than 5% - NOT acceptable

Table 2-5: Case studies on polypropylene pipe (continued)

Reference	Type of pipe	Location	Pipe Size (inch)	Length of pipe (ft)	Trench size (ft)		Type of backfill/ depth of burial(ft)	Applications	Max % of deflection	Joint Separation (inch)	Remarks
					Depth	Width					
Howard (1994)	Fiberglass	Riga, Latvia	36.8	5.5	5.35		Loose sand and clay/9.84 ft	Drainage and irrigation	12.7	-	Max.deflection is greater than 5% - NOT acceptable
Fernando (1992)	Corrugated PVC & Corrugated HDPE	Ohio	18 & 36	9	7		ODOT #310 sandy & ODOT #57 stone and ASTM class III soil	Sewer drainage	19.15,12.7 & 9	-	Max.deflection is greater than 5% - NOT acceptable
Howard (1990)	PVC	Elba, Nebraska	27	20	11 to 13		Poorly graded sand and clay/15 ft	Sewage drainage	9.4	3% of dia	Max.deflection is greater than 5% - NOT acceptable
Remarks	Pipes such as HDPE, PVC, Fiberglass and PP are widely		Size of the pipe varied from 18 to 60 inches	length of the pipe varied from 4.8 to 983.6 ft			Varies type of sand,clay and stone are used	Most of them are used for Sewage drainage & storm water drainage	Max % of deflection varied from 1.5 to 50% for various type of pipes	Joint separation varied from 0.25 to 5.5" for various type of pipes	

2.3 Manholes

2.3.1 Concrete and Brick Manholes Repair and Rehabilitation

The ASCE document on Manhole Inspection and Rehabilitation (Hughes, 2009) identifies several methods for repairing manholes such as coating, lining the surface of manholes and chemical grouting. The document recommends a minimum thickness of coating of at least 0.05 inch. For coatings and liners the document recommends the following ASTM standards for surface preparation:

i. ASTM D 4258 for surface cleaning concrete and brick for coating,

ii. ASTM D 4259 for surface preparation of abrading concrete and brick.

This document also summarizes a couple of more guidance (SSPC SP-13 & ICRI 03732) for the performance and inspection of concrete and brick surface preparation. For polymer coating thickness measurement, it recommends ASTM D 4414 which specifies standard practice for measurement of wet film thickness by Notch gauges. It also recommends ASTM D 4787 test method for continuity verification of sheet linings applied to concrete and brick substrates. For testing pull-off strength of coatings, it recommends (i) ASTM D 4541 (ii) ASTM D 7234.

The EPA document summarizes various products used for manhole rehabilitation (Sterling et al., 2009). However in this report no test method has been recommended to evaluate the performance of the products used.

The WEF manual discusses about coating systems and structural linings used for rehabilitation of manholes (Oman, 2000). It recommends ASTM C 267 test which deals with standard methods for chemical resistance of mortars, grouts and monolithic surfacing and polymer concretes and bricks. Additionally, it recommends ASTM C 321 for testing bonding of coating applied on concrete and brick which is the same method that was used by CIGMAT for the past 15 years. However this manual did not mention any test method for testing the performance of coating under hydrostatic pressure.

2.4 Summary

Based on the review of literature related to material characterization and infrastructure maintenance, the following can be summarized:

1. Various additives were used to alter the setting time of cement slurry and also to improve early compressive strength, flexural strength of the cement mortar.
2. Addition of salt water changed thickening time and early strength of oil well cement.
3. The conductivity of the cement paste was increased by the addition of carbon fiber. Water-to-cement ratio of cement slurry influenced the magnitude of conductive dispersion method.
4. Carbon fiber or silica fume was used to diminish the electric polarization effect.
5. Tensile, flexural strength, tensile ductility, flexural toughness and durability of cement paste were increased by the use of carbon fiber into the cement slurry.
6. For better durability, polypropylene pipe was used for underground drainage.
7. Coating was applied to manholes in order to avoid any possible leakages.

CHAPTER 3

MATERIALS AND METHODS

In this chapter, types of materials and testing methods used in this study are summarized. Sample preparation, setting time test, compression test, method of resistance measurement and the curing conditions are discussed. Full-scale testing of polypropylene pipes joints are also included. Laboratory full scale test on manhole coating materials and the procedure for both concrete and brick manhole were investigated.

3.1 Cement Materials

3.1.1 Mold Calibration

The resistivity (ρ) is defined as RA/L (where, R = measured resistance, A = area of the electrical flow, L = distance between the probe). The two probe test mold was first calibrated using a salt solution of known resistivity. Using the test mold, the resistance of the solution was determined. Then from the resistivity relationship, the A/L ratio of the test mold was determined. This ratio was used to determine the resistivity of the oil well cements.

3.1.2 Electrical Resistivity

The electrical resistance offered by a homogeneous unit cube of material to the flow of a direct current of uniform density between opposite faces of the cube. Also called specific resistance, it is an intrinsic, bulk (not thin-film) property of a material. Resistivity is usually determined by calculation from the measurement of electrical resistance of samples having a known length and uniform cross section according to the following equation, where ρ is the resistivity, R is the measured resistance, A is the cross-sectional area, and l is the length. In the mks system (SI), the unit of resistivity is the ohm-meter.

The formula for electrical resistivity is

$$\rho = RA/l. \tag{3-1}$$

3.1.3 Materials

Oil well cement type class H was used. Portland cement was used for comparison purposes. Effect of setting time due to additives - foam, potassium silicate and carbon fibers were studied. Setting time monitoring was done with resistance and vicat needle test.

3.1.4 Sample Preparation

The preparations of sample were done in accordance with API. For the initial mixing, a high speed propeller-type mixer was used. Carbon fiber of 0.075% was added with the cement mix as base modification. Also various other additives were added with respect to the desired result. Additives were added to the water in the mixer with the mixing intervals of 20 s at 4000 rpm. Cement, water and additive were mixed at the speed of 4000 rpm for 3 min and 35 s at the speed of 1200 rpm. The water/cement ratio in all formulation in this study was 0.4. Four wires were placed into the oil well cement specimen. On each side of the specimen, two wires were placed. The distances between any two wires are same. In order to have consistent result, three specimens were prepared for each specification.

For setting time monitoring testing and compressive piezoresistivity testing, cylinders with diameter of 2 inch were prepared. Conductive cables were embedded in the specimen at the time of casting. For better result, two-probe method was chosen. The spacing between cables was 2 inches. Embedment depth of the conductive part was 1 inch. Also studies showed that the contact resistance was around 1% of the bulk resistance.

3.1.5 Setting Time Test

The Vicat setting test (ASTM C191) was used to determine the initial and final setting times for hydrating cementitious mixtures. It measures the change in the penetration depth of a plunger with a diameter of 1.13±0.05 mm under a constant applied load (300 g) as increasing structure formation acts to reduce the extent of penetration into the specimen. The test identifies initial and final sets at penetration depths of 25 mm and 0.5 mm respectively.

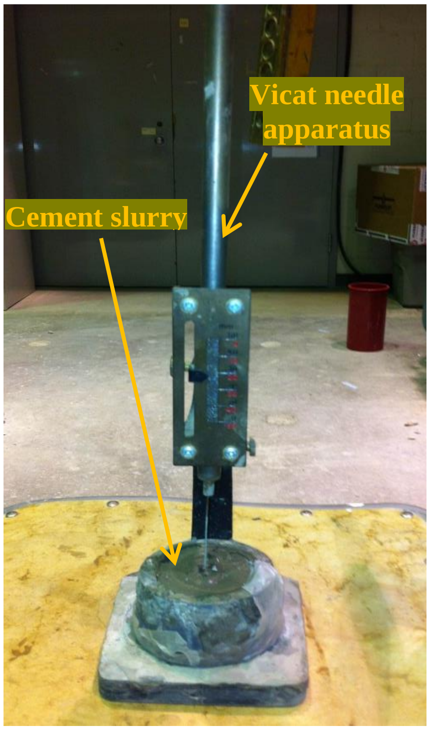


Figure 3-1: Setting Time Test – Vicat Needle Apparatus

3.1.6 Resistance Measurement

The standard ohm-meter was used to measure the resistance between any two wires. The resistance was measured for 21 hours until cement was completely hardened.

Hewlett Packard 34420A NanoVolt/Micro Ohm meter is having a least count of 1 μΩ for electrical resistance measurement. After 10 minutes of specimen made, the resistance measurement was started recording once in 30 minutes for the first 3 hours and once in an hour until specimen hardened completely. The ohm-meter with specimen is shown in the Fig. 3-2.

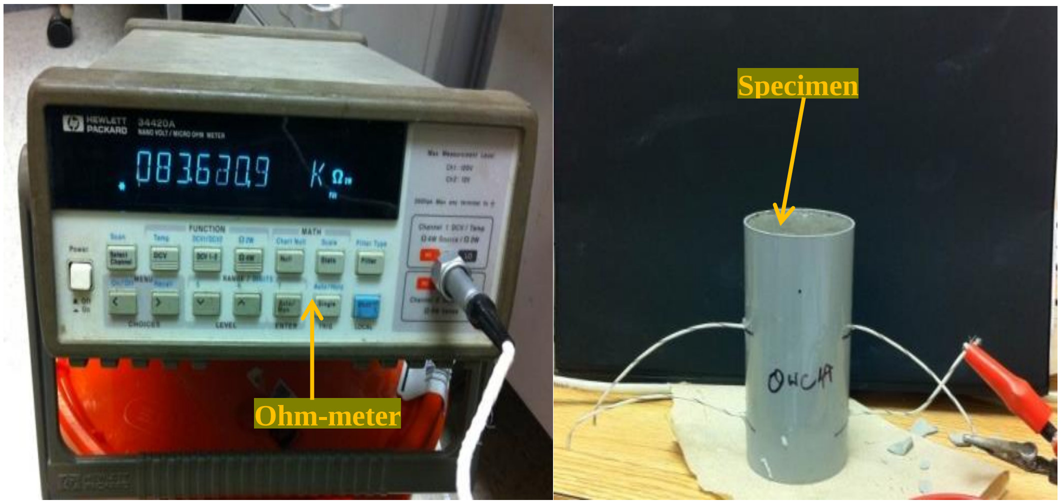


Figure 3-2: Ohm-Meter with Oil Well Cement Specimen

3.1.7 Curing Conditions and Temperatures

Specimens were air cured under the room temperature of 23±2˚C.

3.1.8 Compression Test (ASTM C39)

Uniaxial compression testing set up is shown in Fig. 3-3. The cylindrical specimen was placed at the center of the circular loading plate and uniaxial compression load was applied on the specimen at a predetermined loading rate. Ultimate compression load of the specimen could be determined from the test.

The dimension of the specimen was measured using Vernier caliper and entered into the computer attached to the machine. The loading graph was digitally saved using computer attached to the compression machine. In order to measure the strain, external strain gauge with read out box was used.

Figure 3-3: Uniaxial Compression Test - Experimental Set Up

3.2 Testing Program Polypropylene Pipe Joint

Two instrumented test stands were available at the CIGMAT laboratory, University of Houston (http://cigmat.cive.uh.edu). Each test stand was capable of accommodating two three-foot lengths of 30-inch or greater diameter pipe joined together for testing. Provisions were made to constrain the pipe from moving laterally. The loading points were instrumented with 15,000 pound load cells to measure the applied and reaction loads (Figs.5-1 and 5-2). Test stand provisions will also allow the pipe-joint to be tested under deflection and shear load in accordance with the test protocol. The pipe-joint was first tested under no load followed by the shear test and angular test.

Since water leakage can occur under several joint conditions, three model tests were proposed to closely represent the field situations. In all the cases, after loading, infiltration was tested with a hydrostatic pressure up to 7 psi. Tests were performed in duplicates resulting in six model tests for each pipe joint.

3.3 Manholes

3.3.1 Concrete Manhole

A 4 feet diameter and 3 feet high concrete manhole was used for this study. The thickness of concrete manhole was 3.125 inch. Schematic picture of the concrete manhole was shown in Fig. 3-4 & 3-5. The manhole had a joint in the middle which was leaking.

The coating material consists of three parts as shown in Fig. 3-6. Polyurethane layer was in the middle which was covered by polyuria coatings on top and bottom sides.

There are twelve steel rods were placed in the test chamber which were acted as a reference point for measuring the movement of coating. Also it protected the coating material from excessive swelling due to a high hydrostatic pressure.

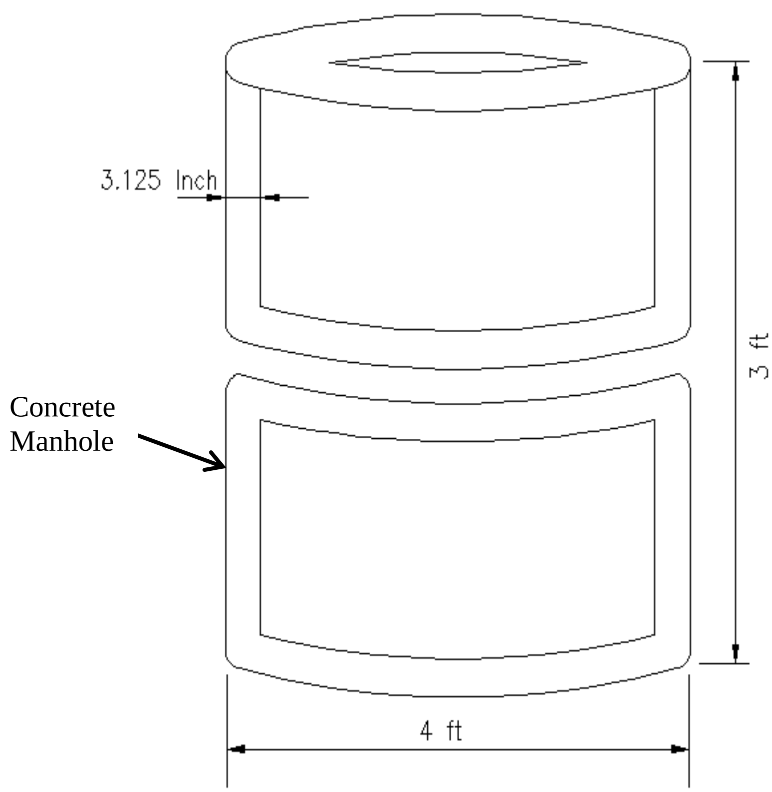
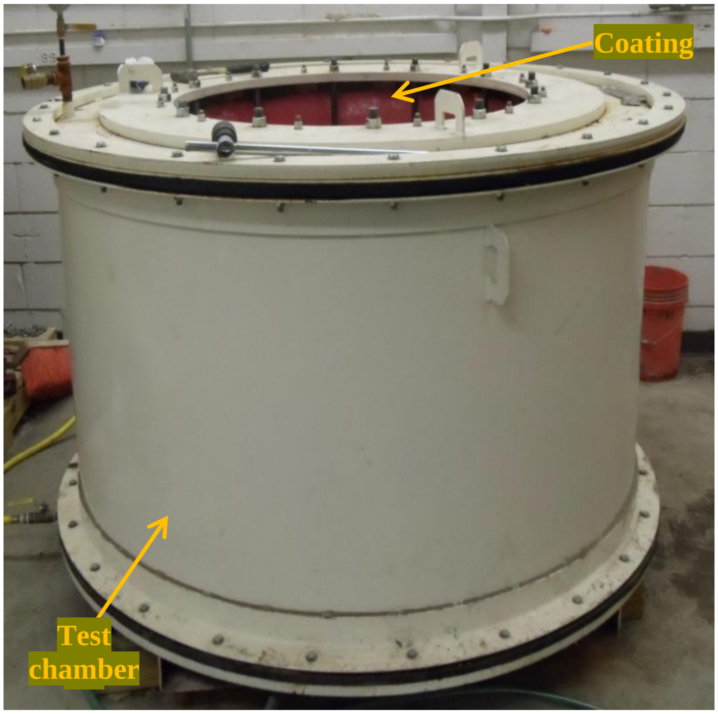


Figure 3-4: Schematic of the Concrete Manhole



(a) Elevation

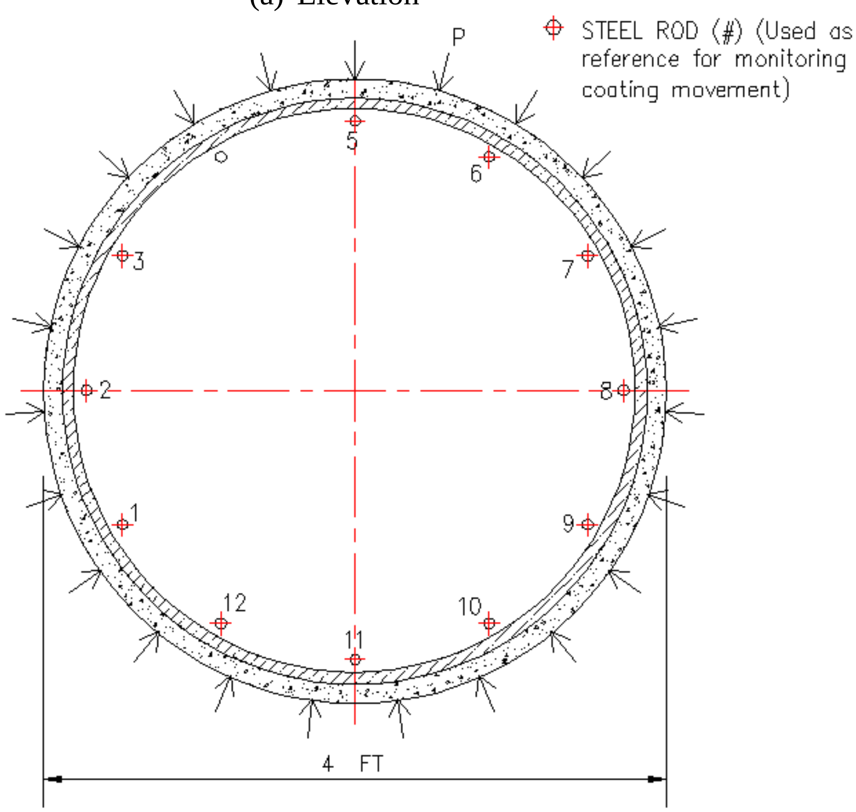


Figure 3-5: Test Chamber

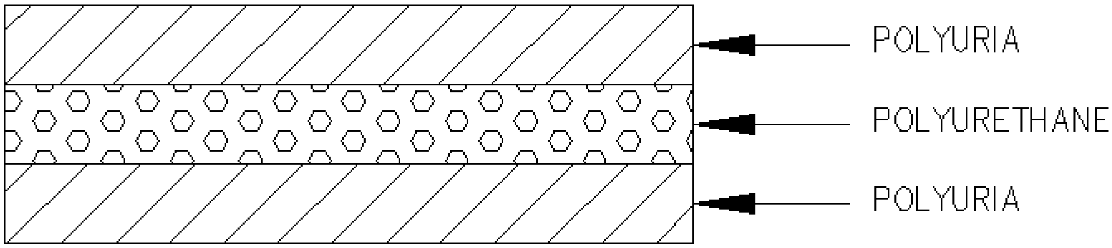


Figure 3-6: Cross Section of the Three Layer Coating

3.3.1.1 Full-Scale Test

The coating was applied to a wet concrete surface. Wet coating condition simulates a manhole in service:

a) Saturation of manhole: The concrete manhole was submerged into a water tank (Fig 3-7) for 7 days before applying the coating.

Figure 3-7: Saturation of Manhole

b) Wet Coating

The concrete surface was water blasted before coating with Spectrashield coating. The coating was done by the coating manufacturer.

c) Pressure Test

The coated concrete manhole was placed in a specially designed test chamber for testing manholes (Fig 2). The hydrostatic pressure was applied from outside the manhole in incremental steps to the coated concrete manhole. The coating was inspected for leaks and the distance between the steel rod and coating were measured to an accuracy of 0.001".

3.3.1.2 Laboratory Test

a. Bonding Strength

Bonding tests were performed to determine the bonding strength between the concrete and the Spectra Shield coating material.

CIGMAT CT-3: In this test the coating was sandwiched between a pair of rectangular concrete block specimens and then tested for bonding strength (Fig. 3-8). Wet specimens were used to simulate the extreme coating conditions. The bonded specimens were cured under water up to the time of testing. Total of three tests were performed.

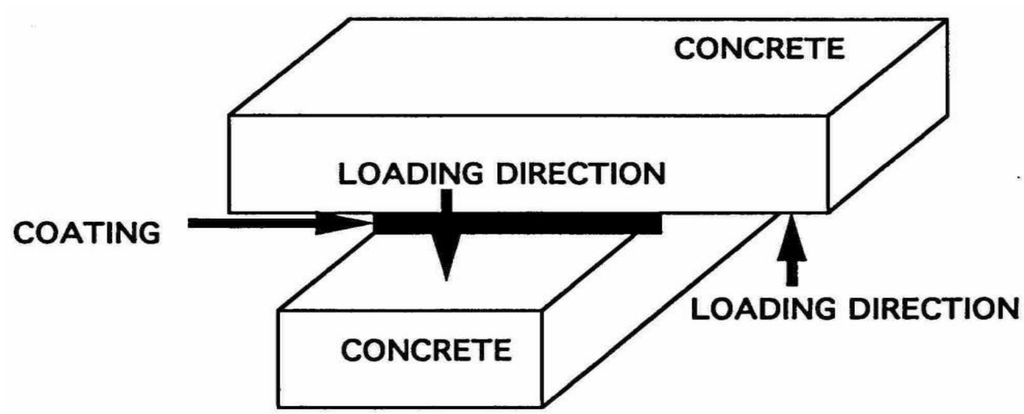


Figure 3-8: Schematic of CIGMAT CT-3 Bonding Test

Specimen preparation:

The coating was applied between a pair of rectangular wet concrete blocks.

3.3.2 Brick Manhole

A 4 feet diameter and 3 feet high brick manhole was used for this study (Fig. 3-9 & 3-10). The manhole had a joint in the middle which was leaking. The coating material consisted of three parts as shown in Fig. 3-11.

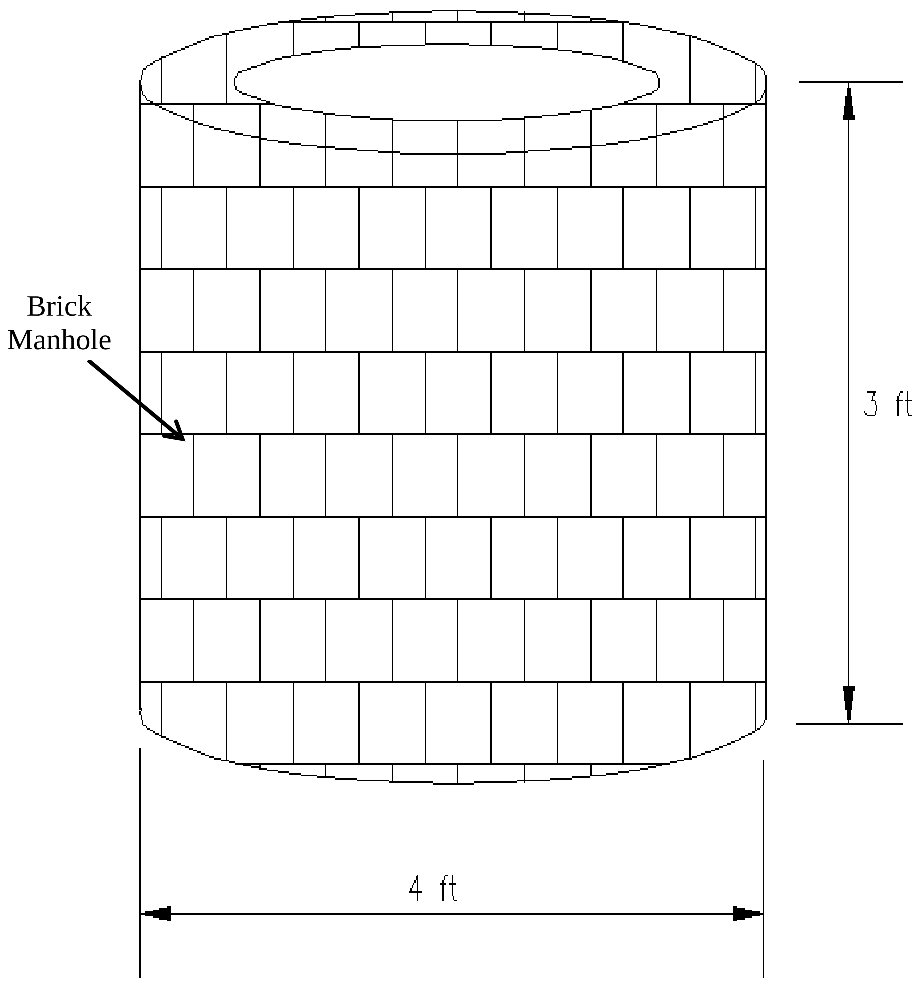
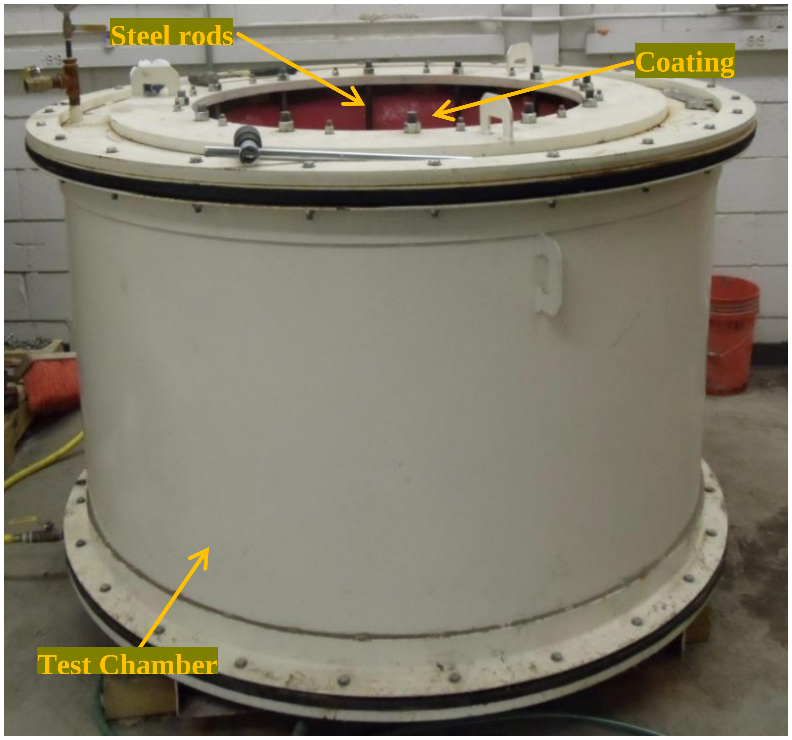
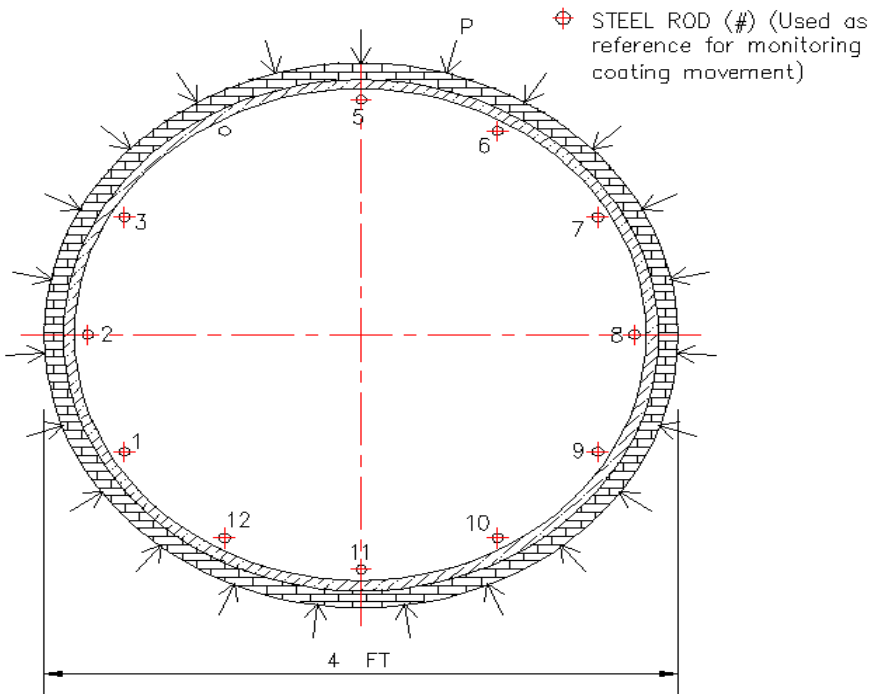


Figure 3-9: Schematic of the Brick Manhole



(a) Elevation



(b) Plan

Figure 3-10: Test Chamber

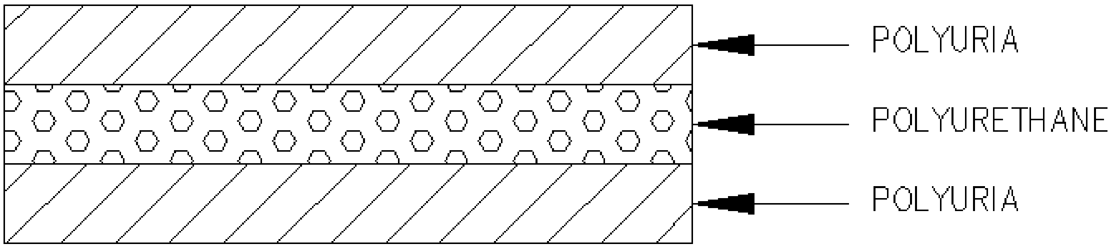


Figure 3-11: Cross Section of the Three Layer Coating

3.3.2.1 Full-Scale Test

The coating was applied to a wet brick surface. Wet coating condition simulates a manhole in service.

a) Saturation of Manhole

The brick manhole was submerged into a water tank (Fig. 3-12) for 7 days before applying the coating.

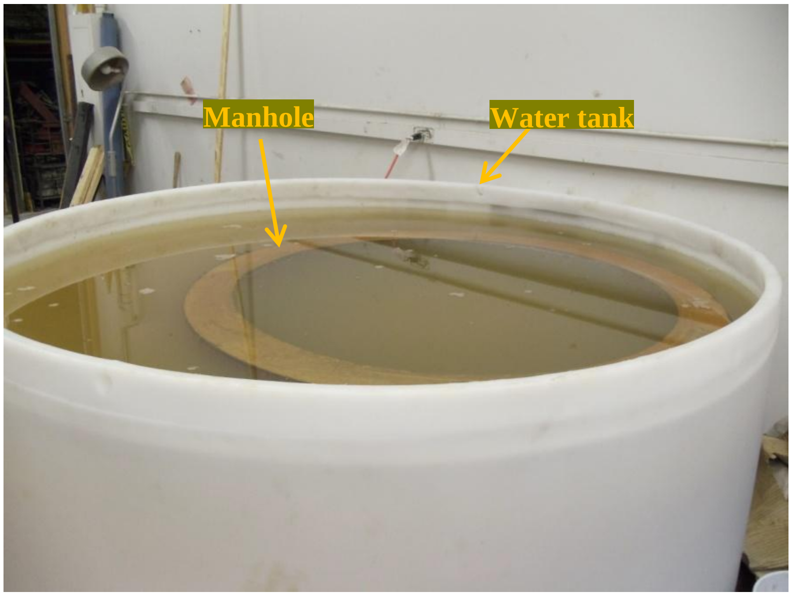


Figure 3-12: Saturation of Manhole

b) Wet Coating

The brick surface was water blasted before coating it with Spectrashield coating. The coating was done by the coating manufacturer.

c) Pressure Test

The coated brick manhole was placed in a specially designed test chamber for testing manholes (Fig. 3-10). The hydrostatic pressure was applied from outside the manhole in incremental steps to the coated brick manhole. The coating was inspected for leaks and the distance between the steel rod and coating were measured to an accuracy of 0.001 inch.

3.4 Summary

Based on the above discussions following conclusions are advanced:

1. Vicat needle apparatus and Tinius olsen machine were used to perform setting time test (ASTM C191) and compression test of oil well cement slurry.
2. For measuring the resistance, ohm-meter was used under the room temperature of 23±2ºC.
3. Straight pipe test, angular test and shear load test were performed in order to evaluate the pipe joint of the polypropylene pipe.
4. Manholes were submerged into the water and manholes were subjected to water blasting before applying the coating materials.
5. The coating material was tested under hydrostatic pressure up to 11 psi for any leakages.

CHAPTER 4

CHARACTERIZATION OF MODIFIED OIL WELL CEMENT

The main purpose of this chapter is to modify oil well cement with carbon fiber, silicate and foam in order to monitor the chemi-resistive behavior of oil well cement during setting. Portland cement was also modified and investigated for comparison purposes. Also compressive piezoresistive behavior of modified oil well cement materials was investigated as the sensing property.

4.1 Change in Density

The densities of each specimen were measured at 0 hours, 8 hours and 21 hours after the specimen made. From the Table 4-1, density of 5% foam oil well cement slurry was 11% less than the normal oil well cement. Whereas density of 5% silicate was 2% higher than the density of normal oil well cement. This difference was because of the higher density of silicate and lower density of foam added to the oil well cement.

4.2 Effect of Carbon Fiber

The specimens of size 2 in. x 4 in. with water to cement ratio of 0.4 were prepared as explained in chapter 3. Carbon fiber of 0.075% was used as an admixture. Using Vicat needle test, the initial and final setting time were found to be 6 hours and 7 hours respectively for water-cement ratio of 0.4.

As shown in Fig. 4-1, the initial, final setting and cement hardening was indicated by the variation in resistivity value. Initially the resistivity was reduced but continued to increase thereafter. Decrease in resistance in the initial stage is because of the mobile ions in cement

(Na^+,K^+,Ca^{2+},SO_4^{2-}) solve in water and form a conductive solution. Also minor peaks in the resistivity could be observed during initial and final setting time.

Table 4-1: Densities of oil well cement (class H) with different additives (kg/m^3)

Time in hours	0% Carbon fiber	0.075% Carbon fiber	1% Foam	5% Foam	Port-land cement	1% Silicate	5% Silicate	Remarks
0	2019	2026	2000	1805	1992	2032	2064	Densities are varied from 1805 to 2064
Remarks	Water loss is higher than with carbon	Carbon fiber prevented extra loss of water	Water loss is very high	This has the lowest density	This density is lower than oil well cement	Water loss is higher than normal oil well cement	It has the highest initial density.	

*Percentage of foam is based on weight of cement

*Percentage of silicate is based on weight of cement

After the cement slurry completely hardened, the resistivity increased by 400 to 500% times the initial resistance. Hence in order to ascertain the cement setting time, chemi-resistivity method was 500% sensitive for 0.075% carbon fiber oil well cement.

After 21 hours of hydration, because of the separate reaction of tri-calcium silicate and di-calcium silicate, calcium silicate hydrate and calcium hydroxide was formed. In the resulting product, calcium silicate hydrate contributed 70% of hydrated cement. Due to the formation of great amount of C-S-H, the resistivity of cement slurry was increased after 21 hours which is shown in the Fig. 4-2.

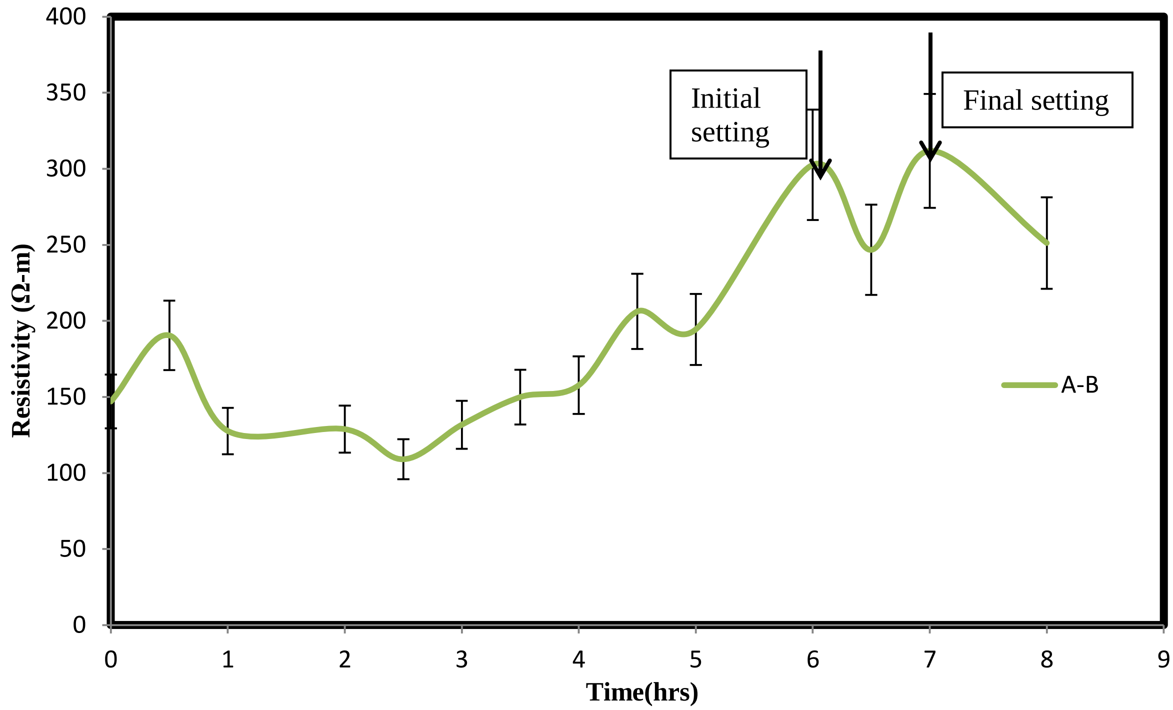


Figure 4-1: Resistivity with Time Relationship for 0.075% CF Modified Oil Well Cement H

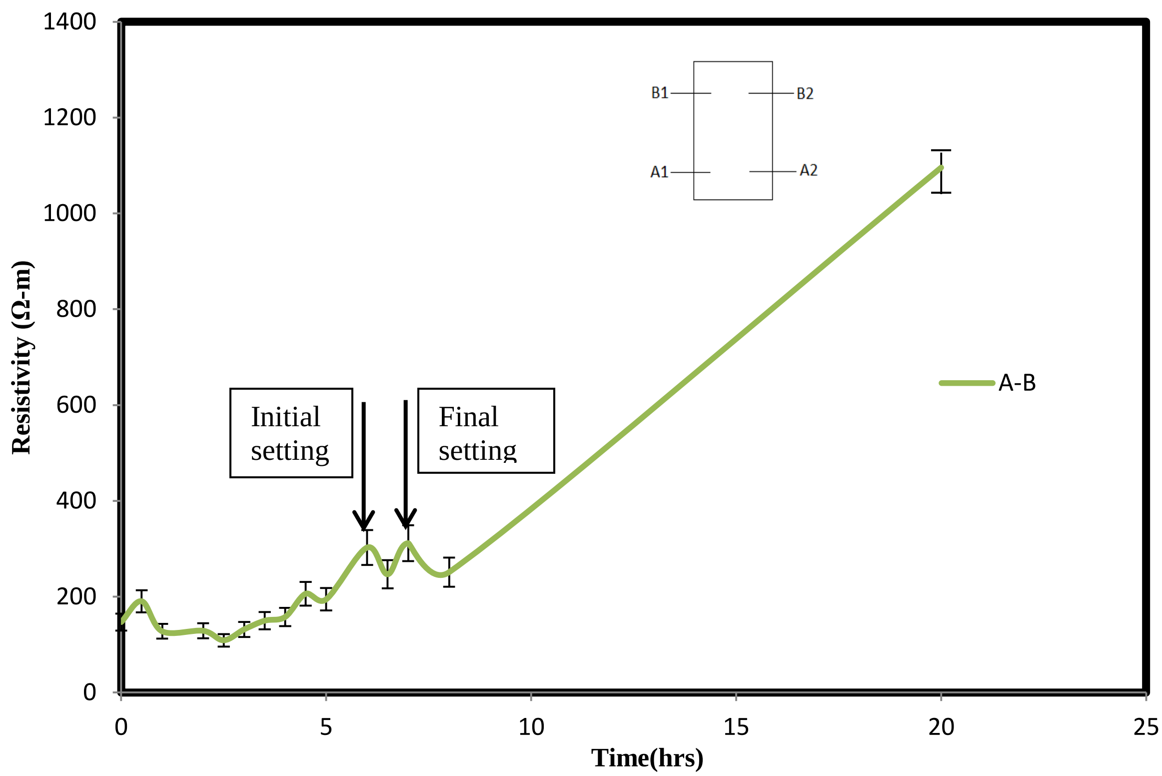


Figure 4-2: Resistivity with Time Relationship for 0.075% CF Modified Oil Well Cement H

The resistivity variation of 0% carbon fiber content with water to cement ratio of 0.4 during setting is shown in the Fig. 4-3. Because of absence of carbon content, the conductive phenomenon of the specimen was lost and became unstable. The fluctuations of resistivity at a given time were high. The value of resistivity was fluctuated between 800 Ω-m to 1200 Ω-m.

Also the hardening behavior of oil well cement was not able to be captured through resistivity because of the absence of a carbon fiber matrix. After one day of specimen made, when the oil well cement completely hardened, the value of resistivity was still fluctuated from 800 Ω-m to 1200 Ω-m without any stability. Hence it is clear that the increment of resistivity after the oil well cement hardened was due to the addition of carbon fiber.

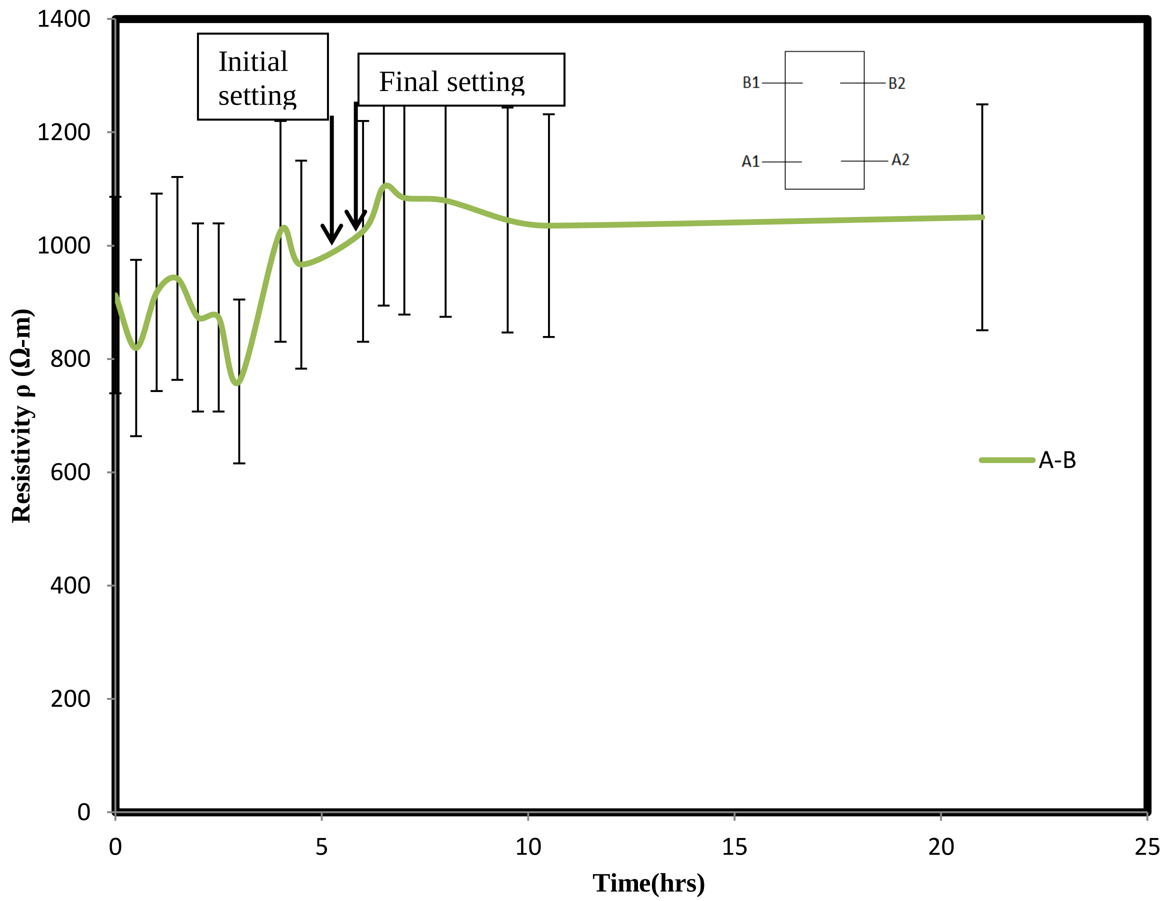


Figure 4-3: Resistivity with Time Relationship for 0% CF Modified Oil Well Cement H

4.3 Effect of Potassium Silicate

Potassium silicate of 1% and 5% were added to the oil well cement along with carbon fiber of 0.075%. Liquid Silicates are converted from a liquid to a solid by the removal of water. As water evaporates, liquid silicates became progressively tackier and more viscous (PQ Corporation, 2012). During the initial period of hydration of cement slurry, the resistivity value of specimen was around 1000 Ω-m which was similar to the resistivity value of no fiber content specimen.

As the dehydration continued, the viscosity of silicate became too high, which resulted in a hardened condition at an early stage. With the addition of 5% potassium silicate, initial setting time was decreased to 4.67 hours and final setting time was decreased to 5.08 hours when tested using the vicat needle test. Also after setting, the value of resistivity was decreased to 10% that of the initial resistivity for 5% potassium silicate content as shown in the Fig. 4-5. For 1% of potassium silicate, the resistivity value was decreased around 40% of its initial after setting as shown in Fig. 4-4.

Therefore the hardening of the cement slurry was ascertained by 10% decrement of resistivity value with 5% potassium silicate content. Also for 1% potassium silicate cement slurry, the hardening nature of cement slurry could be sensed by 40% decrement of resistivity value.

4.4 Effect of Foam

Foam of 1% and 5% by weight of cement were added to oil well cement along with water to cement ration of 0.4 and carbon fiber of 0.075%. Because of light weight nature of foam, the density was decreased by 2.8% for 1% foam and 3.3% for 5% foam addition. The hydration time of foamed cement was very less. As a result, final setting time of 1% foam content cement was 70 minutes and 5% foam content cement was 42 minutes.

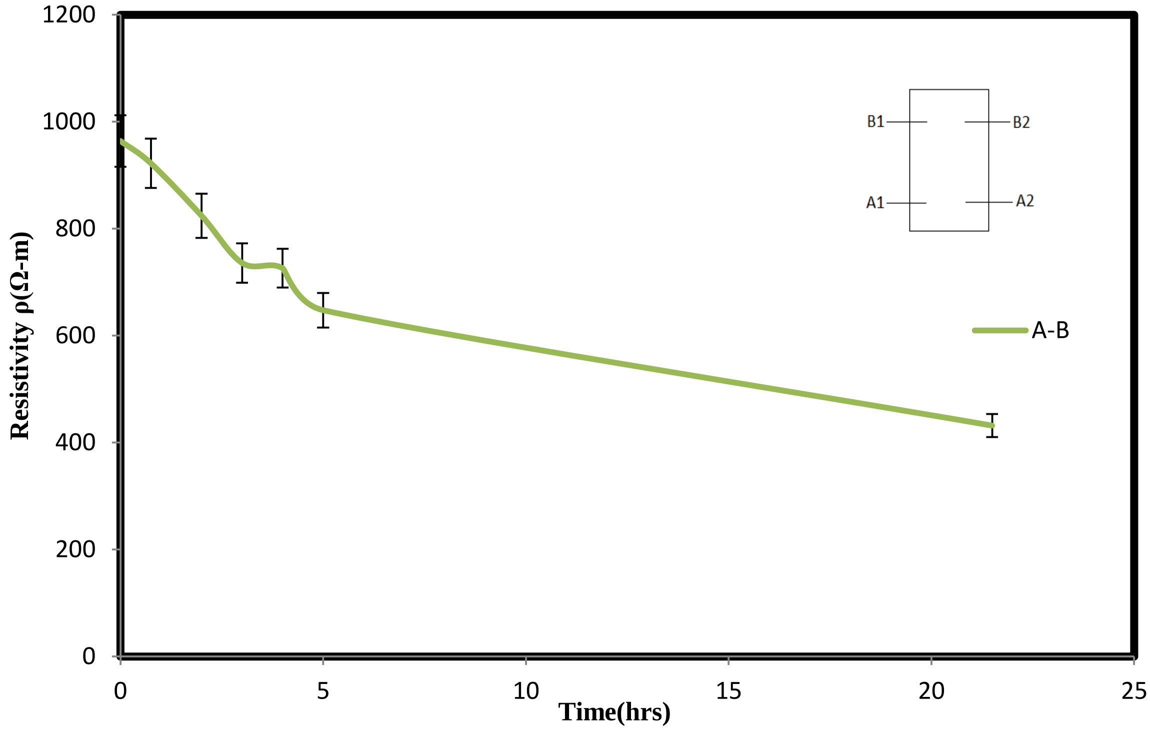


Figure 4-4: Resistivity with Time Relationship for 0.075% CF and 1% K_2SiO_3 Modified Oil Well Cement H

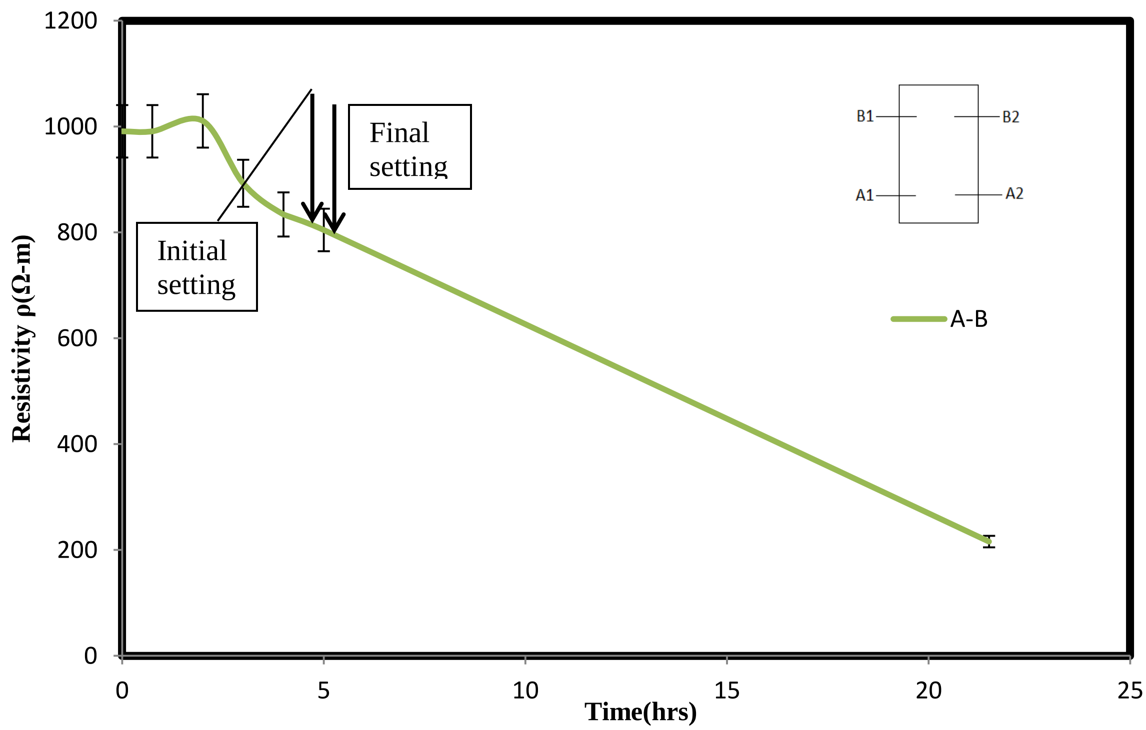


Figure 4-5: Resistivity with Time Relationship for 0.075% CF and 5% K_2SiO_3 Modified Oil Well Cement H

As shown in Fig. 4-6 and 4-7, the initial, final setting and cement hardening was indicated by the variation in resistivity value. Initially the value of resistivity was small but continued to increase thereafter. Decrease in resistance in the initial stage is because of the mobile ions in cement (Na^+,K^+,Ca^{2+},SO_4^{2-}) solve in water and form a conductive solution.

After setting of cement slurry, the resistivity of 5% foamed cement was increased by 11 times the initial resistance. For 1% foamed cement, the resistivity was increased by 35 times the initial resistivity value, a considerable change can be used to monitor the hardening of oil well cement. This change could be attributed to the formation of great amount of C-S-H which was a product of hydration.

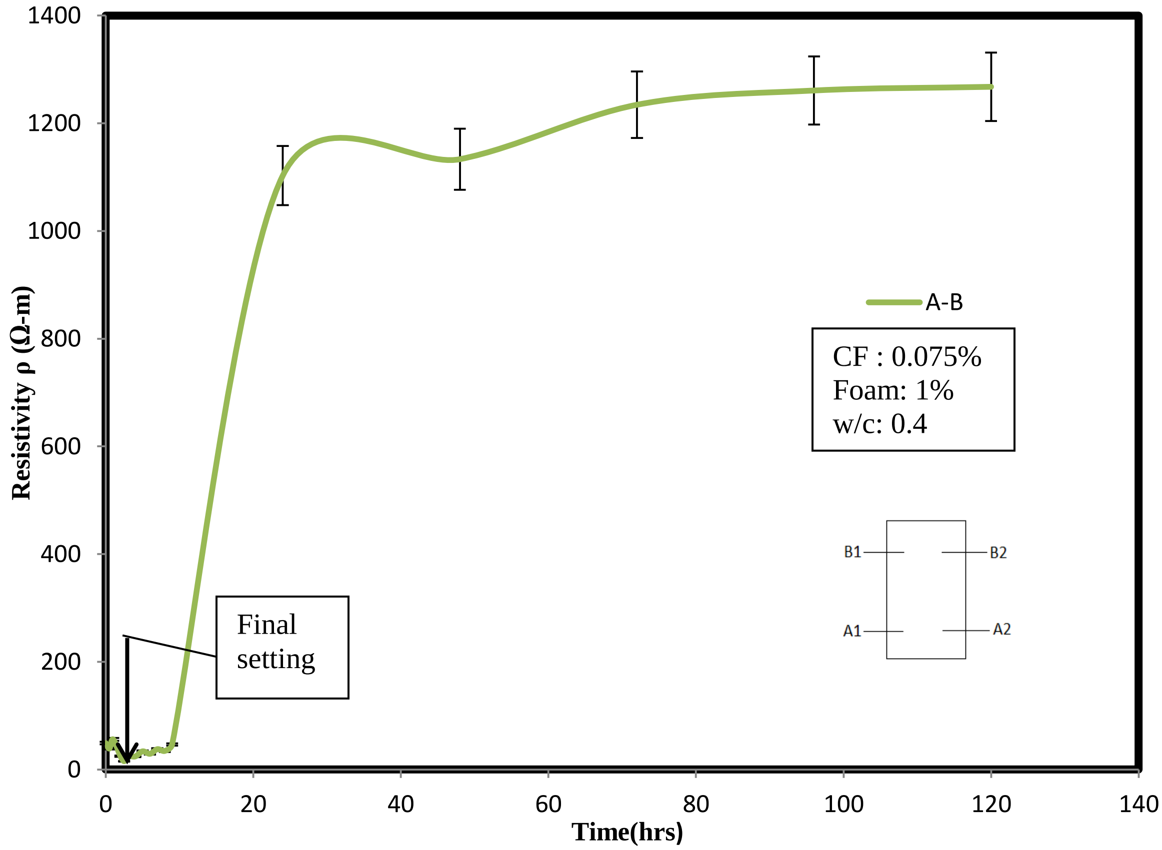


Figure 4-6: Resistivity with Time Relationship for 0.075% CF and 1% Foam Modified Oil Well Cement H

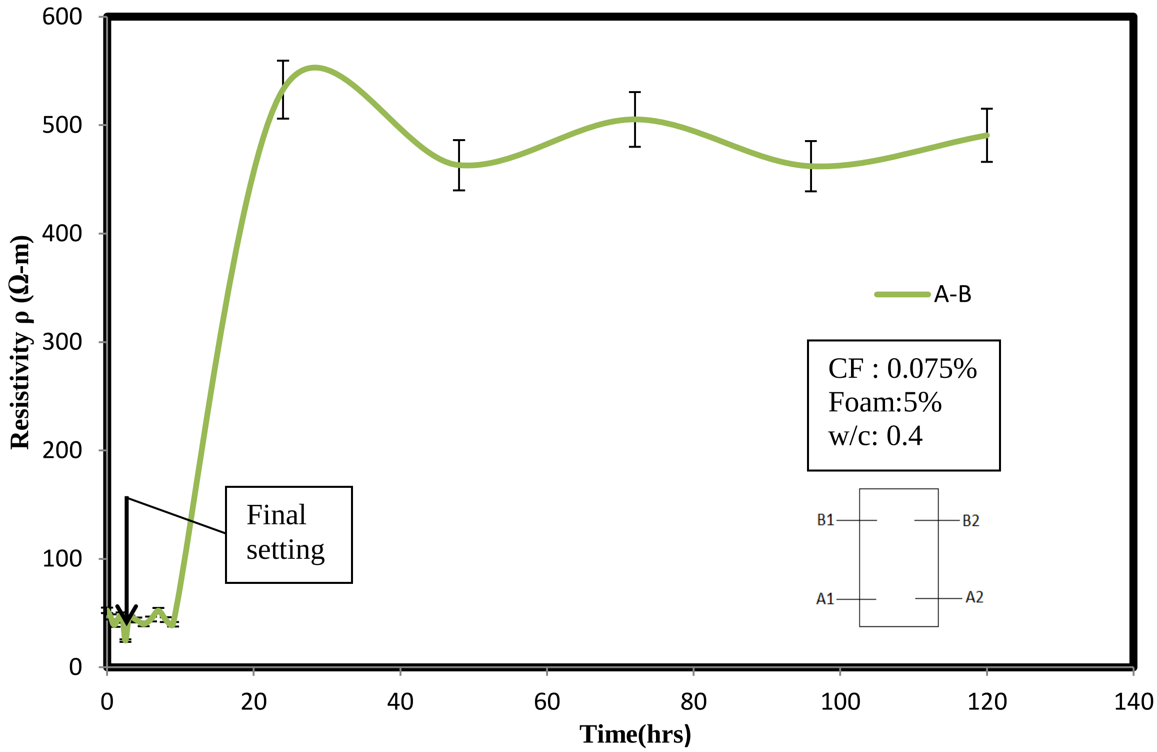


Figure 4-7: Resistivity with Time Relationship for 0.075% CF and 5% Foam Modified Oil Well Cement H

4.5 Setting Time Monitoring on Portland Cement

Portland cement slurry of water to cement ratio of 0.4 with 0% and 0.075% carbon fiber content specimen were prepared. The initial and final setting time were found to be 6 hours and 6.6 hours respectively for water-cement ratio of 0.4 using vicat needle test. As shown in Fig. 4-8 and 4-9, the value of resistivity at the initial stage of hydration was considerably low with carbon fiber of 0.075% while the value of resistivity with 0% carbon fiber was similar to oil well cement. This was the direct effect of function of carbon fiber on resistivity value of cement slurry. In the case of Portland cement, after the cement was hardened the increment of resistivity was low when compared with the oil well cement. Hence it could be ascertained that presence of 2 to 4% gypsum affected the conductive nature of cement slurry considerably. However the resistivity value of 0% carbon content Portland cement was similar to the corresponding oil well cement slurry as it was varied around 800 Ω-m to 1200 Ω-m.

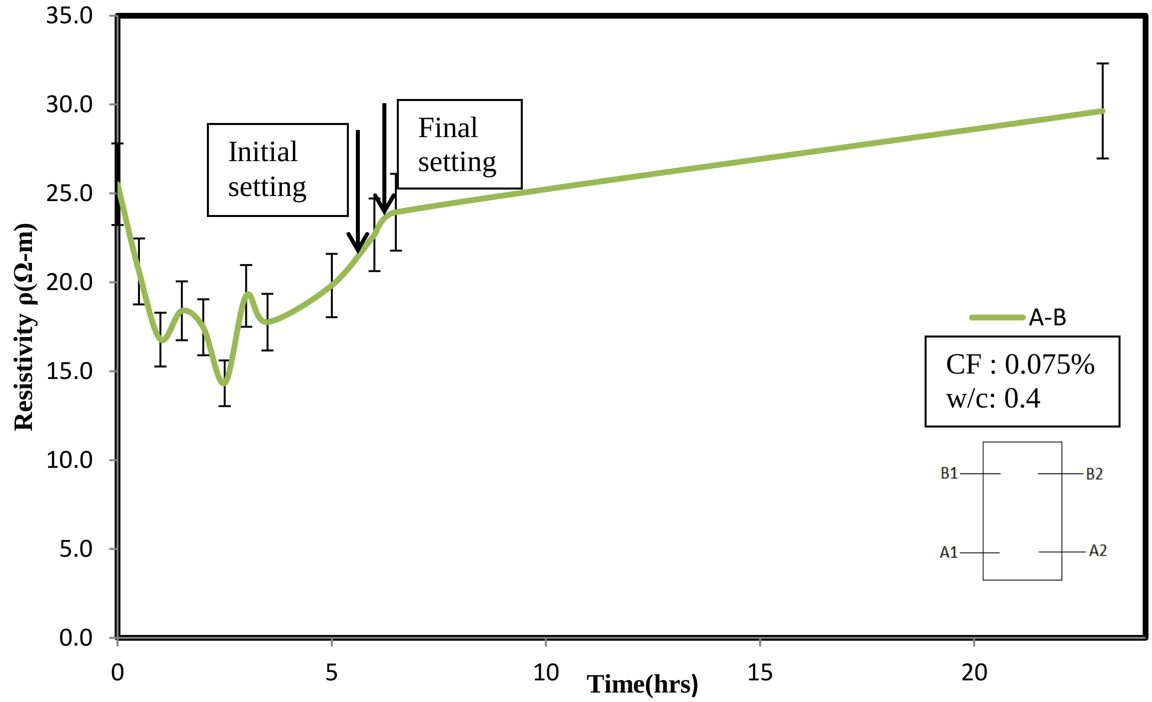


Figure 4-8: Resistivity with Time Relationship for 0.075% CF Modified Portland Cements

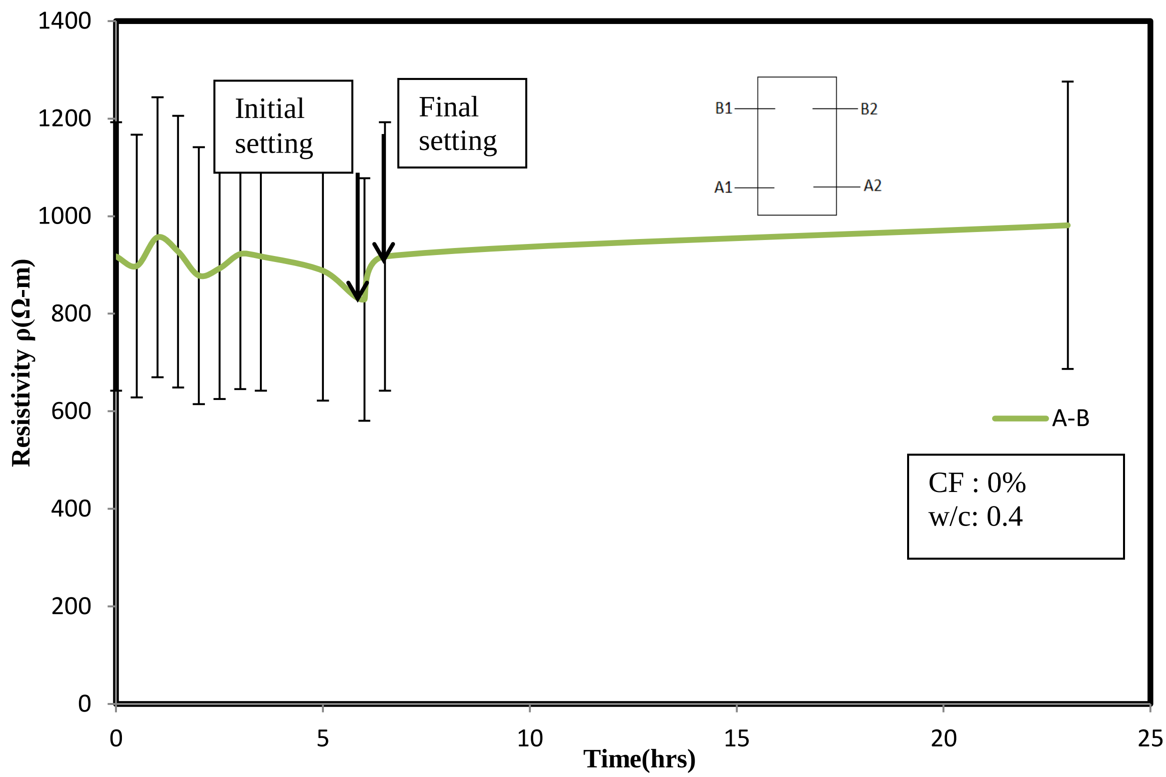


Figure 4-9: Resistivity with Time Relationship for 0% CF Modified Portland Cements

4.6 Piezo-Resistive Behavior

After 7 days of air curing, the specimen with 0.075% carbon content was tested under compression using Tinius Olsen Machine. Both bottom and top sides of the specimen was capped with sulpher for insulation against current.

The change in resistivity was plotted against compression stress in the Fig. 4-10. The change in resistivity was responded well with respect to the change in compressive stress values. For example, the change in resistivity was 45%, for the compressive strength of 5 MPa which was a small stress comparatively.

The percent change of strain for the corresponding failure stress was around 0.2% for cement specimen. Whereas the compressive failure stress of around 20 MPa was sensed with the change of resistivity of 80%. The resistivity changes about 400 times higher than the change in strain. The specimen was tested after 7 days of specimen made.

For 0.125% carbon fiber content specimen, the change in resistivity value was less until the crack was formed. Once the crack formed, the matrix between carbon fibers was broken which increased the change in resistivity value considerably. The specimen was tested after 7 days of specimen made. The compressive stress at the time of failure was around 25 MPa as shown in the Fig. 4-11. The percentage change in resistivity at the time of failure of cement specimen was around 150%.

Cement specimen with 5% of foam and 0.075% of carbon fiber was tested under compression using Tinius olsen machine after 28 days of air curing. At the initial compressive stress values, the change in resistivity value was not high. However after formation of crack in the specimen, the change in resistivity became sensible with respect to the compressive stress values. As shown in the Fig. 4-12, percentage change in resistivity at the failure stress of 35 MPa was around 60% whereas the failure strain percentage was around 0.2% which was a small variation.

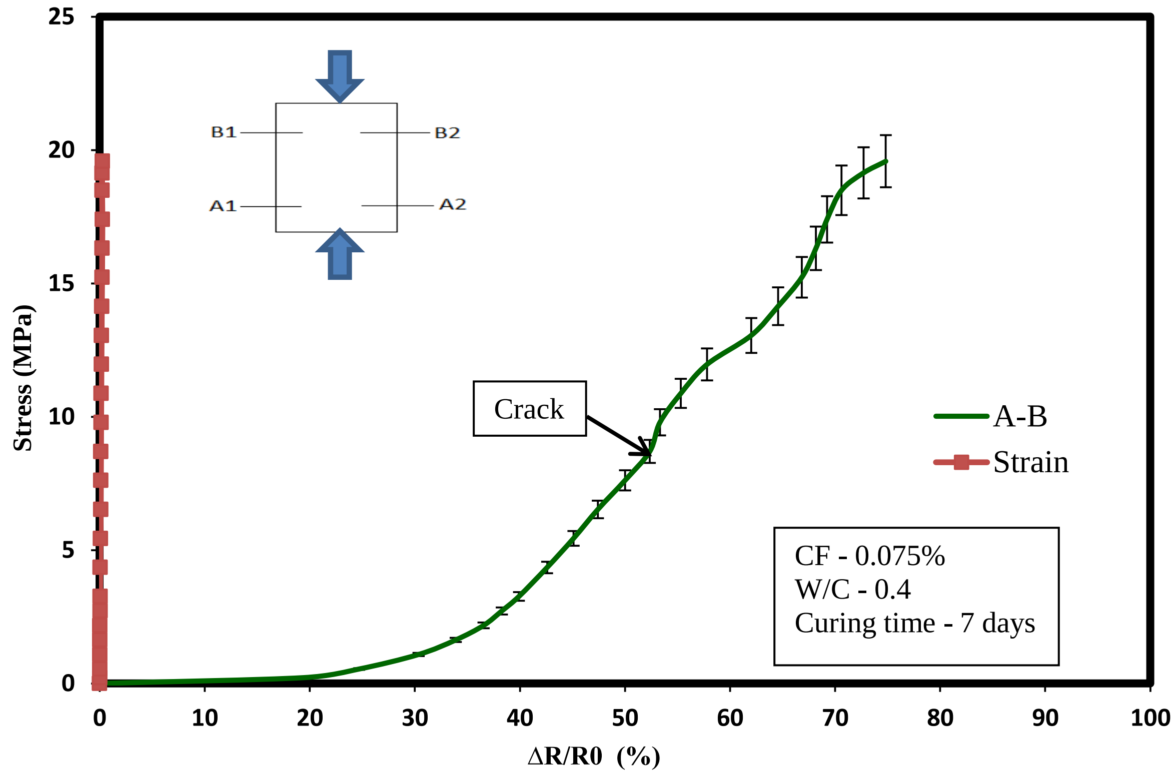


Figure 4-10: Piezo-resistivity Behavior of Oil Well Cement with 0.075% CF

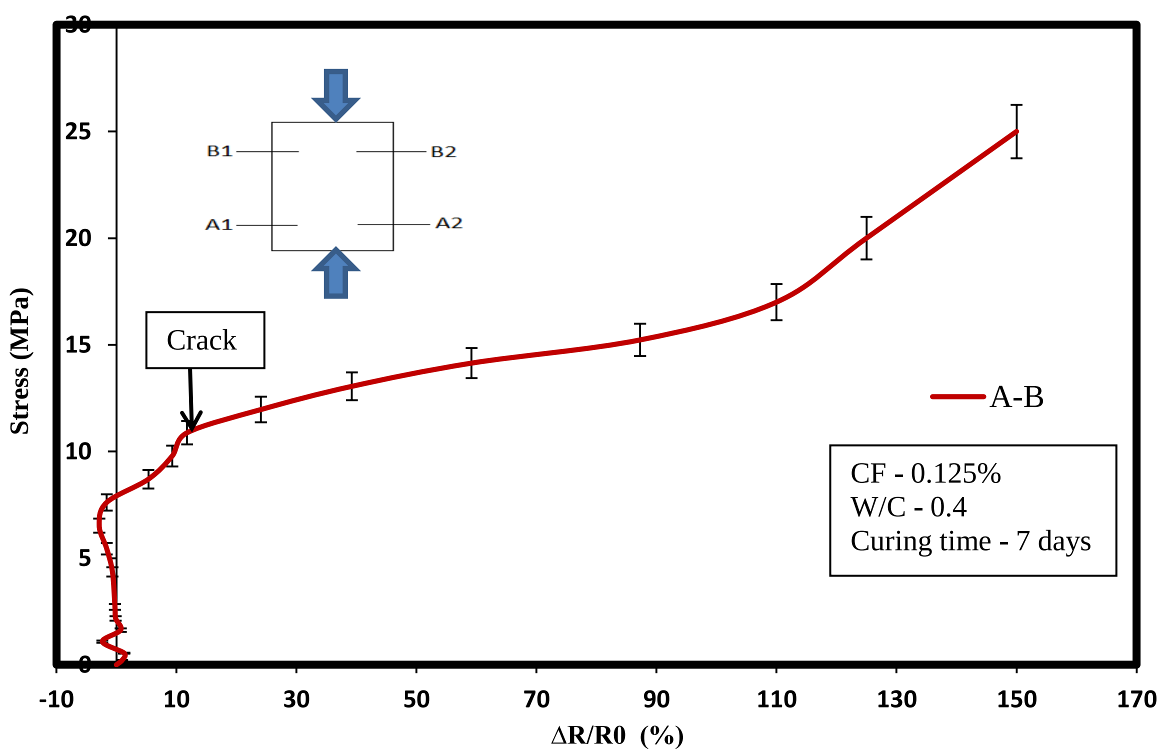


Figure 4-11: Piezo-resistivity Behavior of Oil Well Cement with 0.125% CF

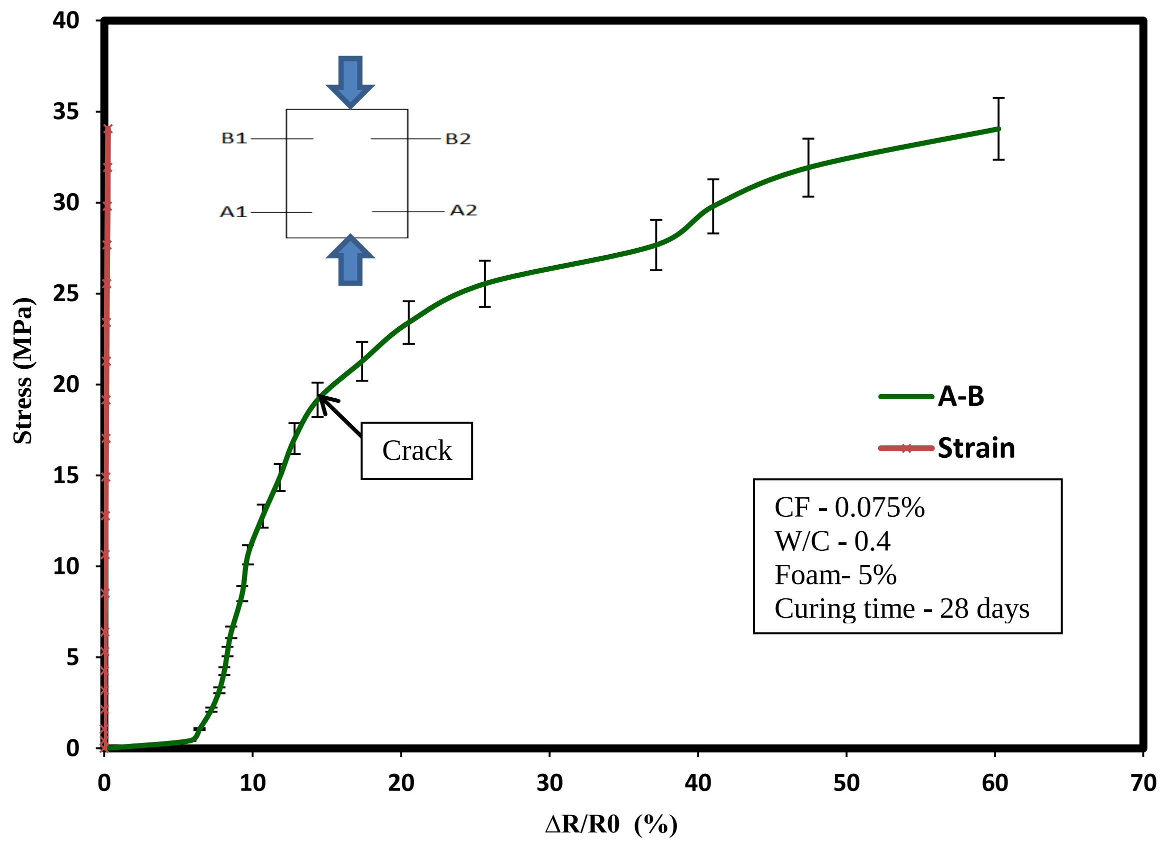


Figure 4-12: Piezo-resistivity Behavior of Oil Well Cement with 0.075% CF & 5% Foam

4.7 Summary

The following conclusions are taken from the above discussions:

1. Oil well cement with 0.075% carbon fiber showed good indications with respect to change in resistivity for: Initial Setting time, Final setting time. After the cement slurry completely hardened, the resistivity increased by 400 to 500% times the initial resistivity. Hence chemi-resistivity method was 500% sensitive than the weight loss method for 0.075% carbon fiber oil well cement. The initial and final setting time were found to be 6 hours and 7 hours respectively

2. The value of resistivity of 0% carbon content cement slurry was still fluctuated from 800 Ω-m to 1200 Ω-m without any stability during and after setting.

3. With the addition of potassium silicate content, during the initial period of hydration of cement slurry, the resistivity value of specimen was similar to the resistivity value of no fiber content specimen. After setting, the value of resistivity was decreased to 40% and 10% of the initial resistivity for 1% & 5% of potassium silicate content respectively. Initial and final setting time was decreased to 4.67 and 5.08 hours for 5% potassium silicate content cement slurry.
4. After setting of cement slurry, the resistivity of 1% foamed cement was increased by 35 times the initial resistance. For 5% foamed cement, the resistivity was increased by 11 times the initial resistivity, a considerable change can be used to monitor the hardening of oil well cement. The density was decreased by 2.8% for 1% foam and 3.3% for 5% foam addition.
5. For Portland cement, the value of resistivity at the initial stage of hydration was considerably low with carbon fiber of 0.075% while the value of resistivity with 0% carbon fiber was similar to oil well cement. This was the direct effect of function of carbon fiber on resistivity value of cement slurry. After the cement was hardened the increment of resistivity was low when compared with the oil well cement. Hence it could be ascertained that presence of 2 to 4% gypsum affected the conductive nature of cement slurry considerably. However the resistivity value of 0% carbon content Portland cement was similar to the corresponding oil well cement slurry as it was varied around 800 ohm-m to 1200 ohm-m.
6. Small stresses (comp.) can be sensed with the addition of carbon fiber (0.075% and 0.125%) and foam (5%). The percentage change of strain at failure was around 0.2%. Whereas the compressive failure stress of around 20 MPa was sensed with the change of resistivity of 80% which was a good indication. Electrical resistivity was proven as a better indicator than strain.

CHAPTER 5

PP PIPE JOINT TESTS

In this study the total of four pipe joints were tested. Tests No.1 & No.2 were conducted on the triple wall pipes (ASTM F 2764 – 11a) with specified stiffness of 46 lb/in/in. Tests Nos. 3&4 were conducted on double wall pipes (ASTM F 2881 - 11) with specified stiffness of 46 lb/in/in. The joints were tested under aligned (straight and shear load tests) and misaligned positions (angular test) (Fig. 5-1 (a), (b) and (c)). The bladders were built to fit the pipe joints using a combination of rubber and plastic sheets (Fig. 5-2). The joints were pressurized under each mode of loading starting at 3 psi hydrostatic pressure. The four joint tests were performed during summer 2012.

5.1 Test No. 1. (PP – Triple Wall Pipe)

5.1.1 METHOD A: Straight Pipe Joint Test (Figure 5-1 (a))

Actual test setup is shown in Fig. 5-2(a). The test results are summarized in Table 5-1. No water leak was observed at the joint during the total test period of 30 minutes with maximum hydrostatic pressure of 7 psi for 10 minutes

5.1.2 METHOD B: Angular Deflection Test (Figure 5-1 (b))

In the angular deflection test, the angles 0.50, 1.00, 1.50 and 2.00 degrees at the joint were tested. The testing time under each angle was 30 minutes with a maximum hydrostatic pressure of 7 psi for 10 minutes. The results of the test are summarized in Table 5-1. The relationship between angle of rotation and shear load at the joint is shown in Fig. 3. Shear force varied from 55 to 533 lbs at the joint. No water leak was observed at the joint during the total test period of 2 hrs.

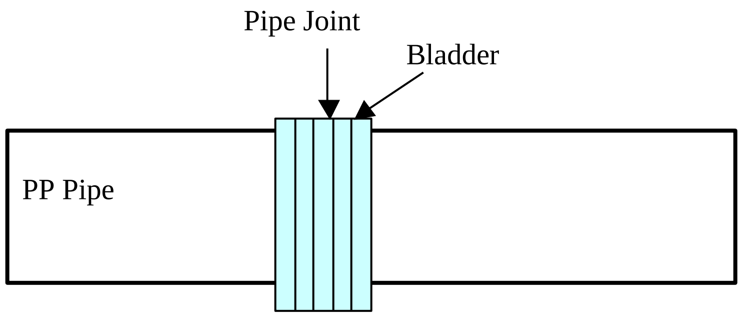
5.1.3 METHOD C: Shear Load Test (Figure 5-1 (c))

The joint was tested under shear loading according to Method C. The load was applied at load cell No. 1 on the pipe and was increased in steps of 500, 1000, 2000, 2500 and 3500 lbs. The test results are summarized in Table 5-2 and the total testing time was 2.5 hours. The Shear load at the joint vs. applied load and the deflection of the pipe at the loading point vs. applied load are shown in Fig. 5-4 and Fig. 5-5 respectively. The maximum shear load at the joint was 2790 lbs. and there was no water leak. The maximum deflection of the pipe occurred at the point of loading (Cross-section 1-1 (Fig. 5-1c)) of the pipe were -1.95% (Extension) and 2.97% (Compression) (based on the pipe diameter) in the horizontal and vertical directions respectively.

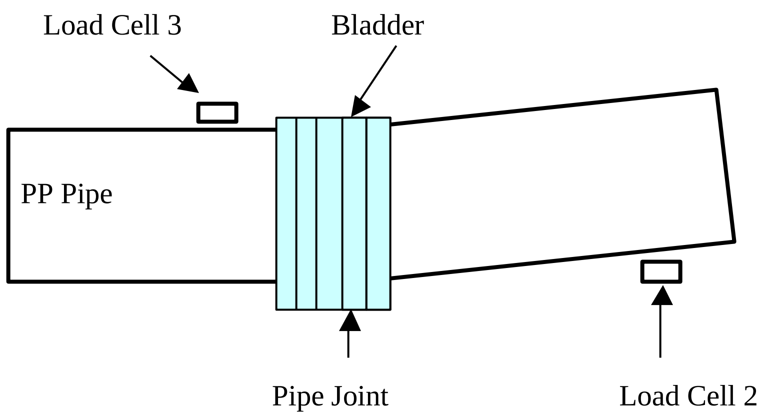
5.1.4 Summary

The composite bladder performed as designed. Total testing time for Method A, B, and C were 0.5 hour, 2 hours and 2.5 hours respectively. No leakage was observed at the tested joint for all the testing conditions. In the shear load test, the maximum shear load at the joint was 2790 lbs. and there was no water leak. The maximum deflection of the pipe occurred at the loading point at maximum shear load (based on the pipe diameter).

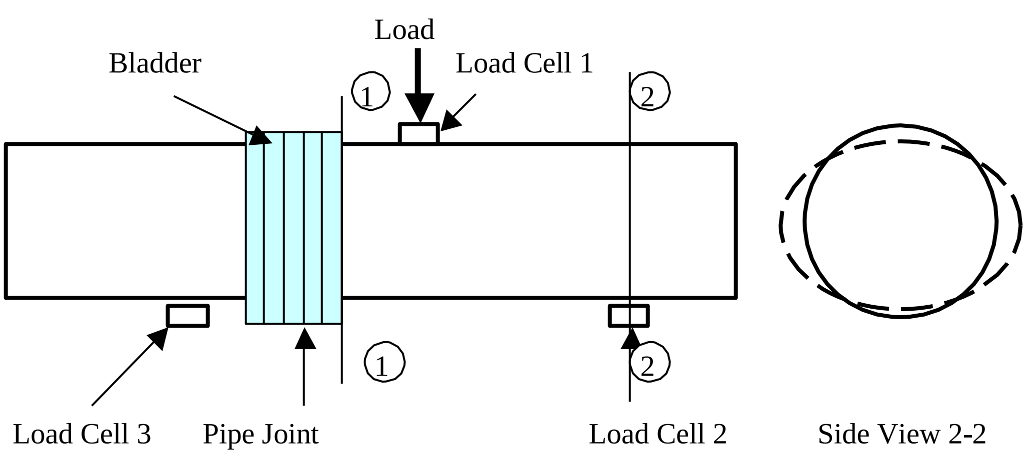
The vertical and horizontal deflections at the loading point (Cross-section 1-1 (Fig. 5-1c)) of the pipe were -1.95% (Extension) and 2.97% (Compression) (based on pipe diameter) respectively.



(a) Method A: Straight Pipe Joint Test



(b) Method B: Angular Deflection Test



(c) Methods C: Shear load Test

Figure 5-1: PP Pipe-Joint Test Configurations

(a) Method A: Straight Alignment Test

(b) Method B: Angular Deflection Test

Figure 5-2: Views of the Triple Wall PP Pipe Joint Tests and Loading Frame

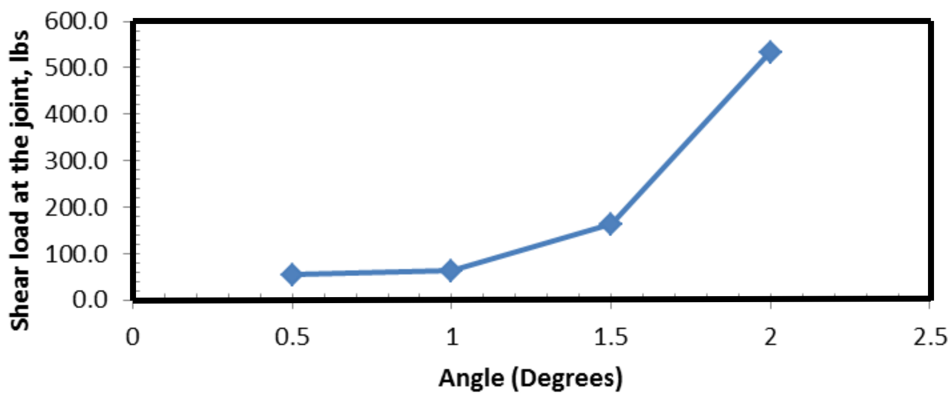


Figure 5-3: The Relationship between Angle and Shear Load in Test No. 1

Table 5-1: Results from Straight and Angular Deflection Test (Test No. 1)

Method	Angle (^{o})	Pressure (psi)	Time (min)	Leakage	Remarks
A	0	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
B	0.5	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
	1.0	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
	1.5	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
	2.0	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
Remark	Up to 2^{o}	3 to 7 psi	Total 2.5 hrs	No Leak	No water leak, Bladder performed as designed

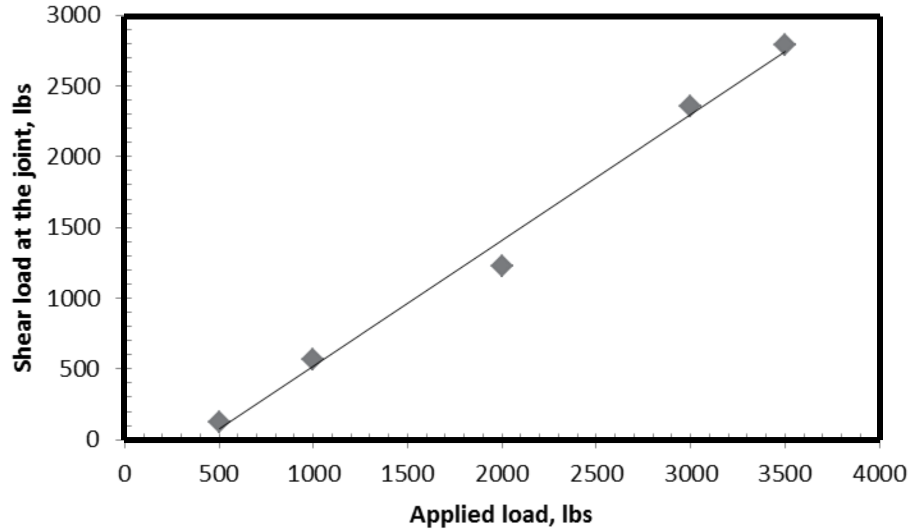


Figure 5-4: Applied Load vs. Shear Load in Test No. 1

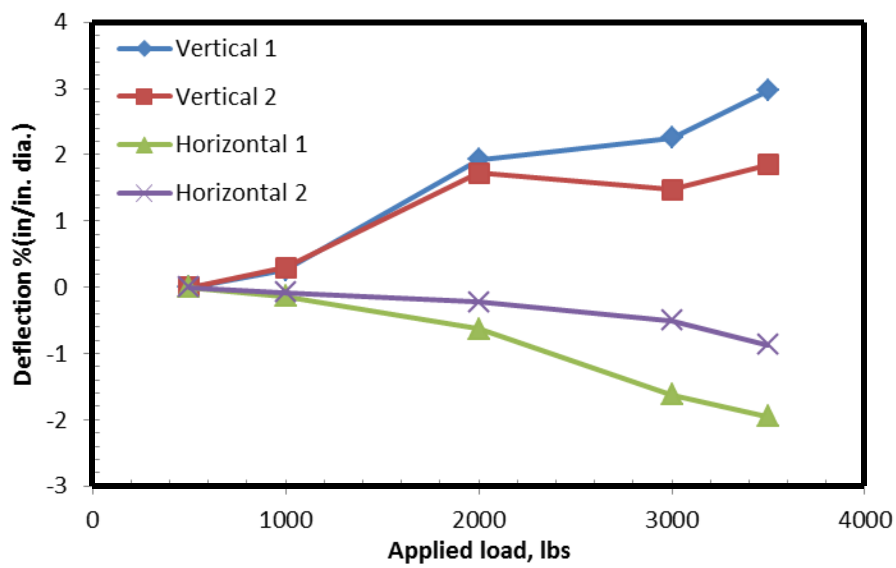


Figure 5-5: Applied Load vs. Deflection at the Loading Point during Test No. 1

Table 5-2: Results from Shear Load Test (Test No. 1)

Intended Load (lb)	Pressure (psi)	Time (min)	Leakage	Actual Load Applied (lb)	Shear Load(lb)	Remarks
500	3	5	No	497	126	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
1000	3	5	No	994	564	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
2000	3	5	No	1815	1227	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
3000	3	5	No	2998	2352	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
3500	3	5	No	3495	2790	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
Remarks	Up to 7 psi	Total 2.5 hrs.	No leak	Maximum load 3495 lb.	Maximum shear 2790 lb	No water leak

5.2 Test No. 2. (PP – Triple Wall Pipe)

5.2.1 METHOD A: Straight Pipe Joint Test (Fig. 5-1 (a))

Actual test setup is shown in Fig. 5-2(a). The test results are summarized in Table 5-3. No water leak was observed at the joint during the total test period of 30 minutes with maximum hydrostatic pressure of 7 psi for 10 minutes.

5.2.2 METHOD B: Angular Deflection Test (Fig. 5-1 (b))

In the angular deflection test, the angles 0.50, 1.00, 1.50 and 2.00 degrees at the joint were tested. The testing time under each angle was 30 minutes with a maximum hydrostatic pressure of 7 psi for 10 minutes. The results of the test are summarized in Table 5-3. The relationship between angle of rotation and shear load at the joint is shown in Fig. 6. Shear force varied from 162 to 292 lbs at the joint. No water leak was observed at the joint during the total test period of 2 hrs.

5.2.3 METHOD C: Shear Load Test (Fig. 5-1 (c))

The joint was tested under shear loading according to Method C. The load was applied at load cell No. 1 on the pipe and was increased in steps of 500, 1000, 2000, 2500 and 3500 lbs. The test results are summarized in Table 5-4 and the total testing time was 2.5 hours. The Shear load at the joint vs. applied load and the deflection of the pipe at the loading point vs. applied load are shown in Fig. 5-7 and Fig. 5-8 respectively. The maximum shear load at the joint was 2870 lbs. and there was no water leak. The maximum deflection of the pipe occurred at the point of loading (Cross-section 1-1 (Fig. 5-1c)) of the pipe were -1.92% (Extension) and 2.25% (Compression) (based on the pipe diameter) in the horizontal and vertical directions respectively.

5.2.4 Summary

The composite bladder performed as designed. Total testing time for Method A, B, and C were 0.5 hour, 2 hours and 2.5 hours respectively. No leakage was observed at the tested joint for all the testing conditions. In the shear load test, the maximum shear load at the joint was 2870 lbs.

and there was no water leak. The maximum deflection of the pipe occurred at the loading point at maximum shear load (based on the pipe diameter). The vertical and horizontal deflections at the loading point (Cross-section 1-1 (Fig. 1c)) of the pipe were -1.92% (Extension) and 2.25% (Compression) (based on pipe diameter) respectively.

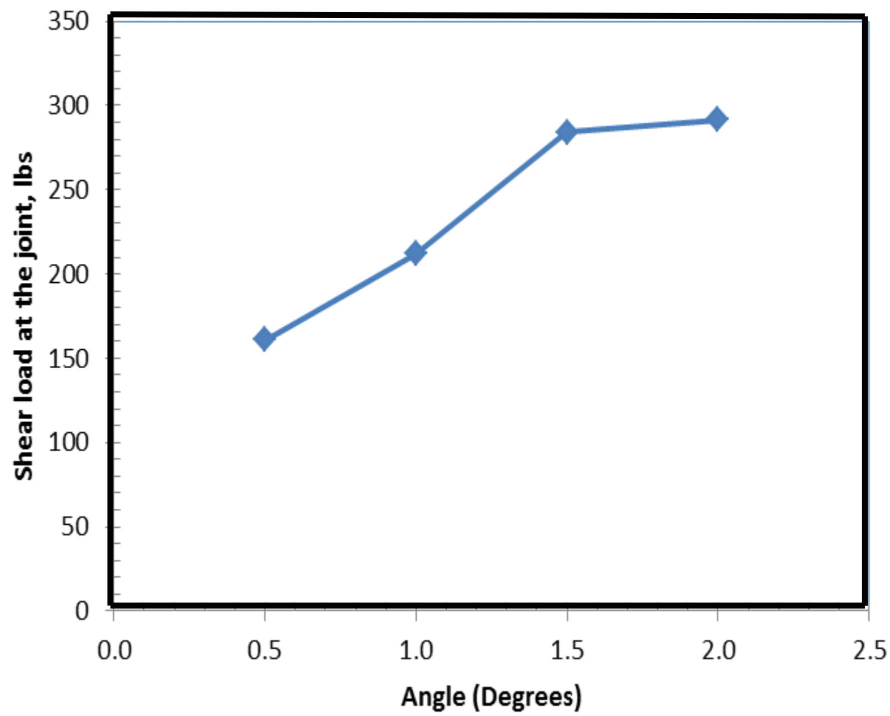


Figure 5-6: The Relationship between Angle and Shear Load in Test 2

Table 5-3: Results from Straight and Angular Deflection Tests (Test No. 2)

Method	Angle (^{o})	Pressure (psi)	Time (min)	Leakage	Remarks
A	0	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
B	0.5	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
	1.0	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
	1.5	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
	2.0	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
Remark	Up to 2^{o}	3 to 7 psi	Total 2.5 hrs	No Leak	No water leak, Bladder performed as designed

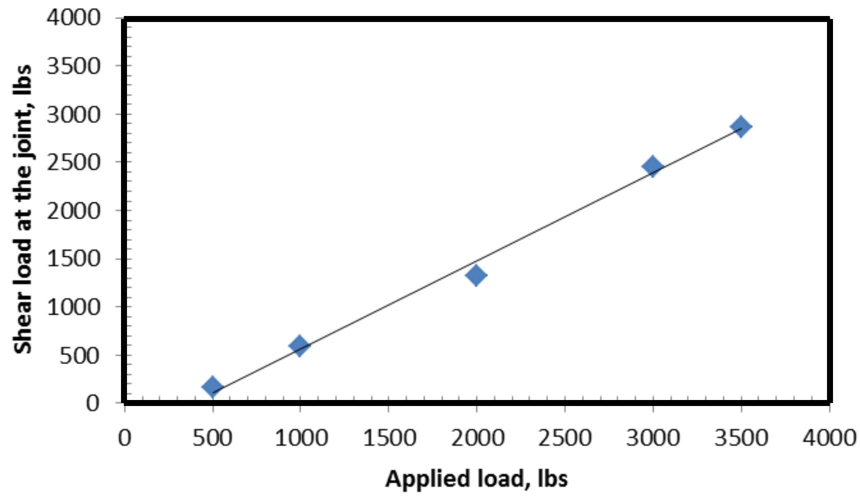


Figure 5-7: Applied Load vs. Shear Load in Test No. 2

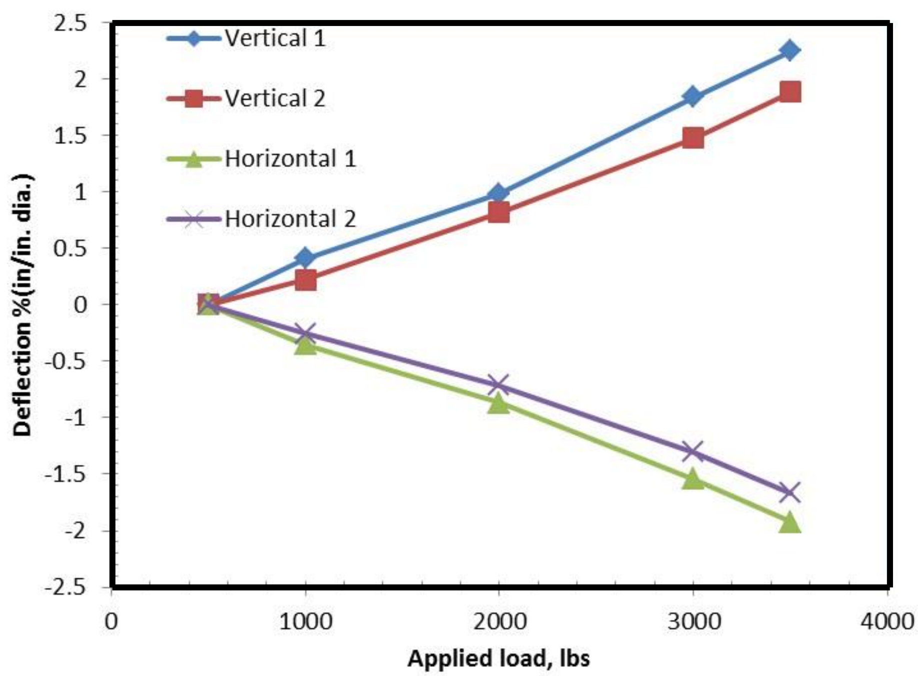


Figure 5-8: Applied Load vs. Deflection at the Loading Point during Test No. 2

Table 5-4: Results from Shear Load Test (Test No.2)

Intended Load (lb)	Pressure (psi)	Time (min)	Leakage	Actual Load Applied (lb)	Shear Load(lb)	Remarks
500	3	5	No	497	162	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
1000	3	5	No	994	593	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
2000	3	5	No	1815	1328	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
3000	3	5	No	2998	2453	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
3500	3	5	No	3495	2870	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
Remarks	Up to 7 psi	Total 2.5 hrs.	No leak	Maximum load 3495 lb.	Maximum shear 2870 lb	No water leak

5.3 Test No. 3. (PP – Double Wall Pipe)

5.3.1 METHOD A: Straight Pipe Joint Test (Fig. 5-1 (a))

The actual test setup is shown in Fig. 5-2(a). The test results are summarized in Table 5-5. No water leak was observed at the joint during the total test period of 30 minutes with maximum hydrostatic pressure of 7 psi for 10 minutes.

5.3.2 METHOD B: Angular Deflection Test (Fig. 5-1 (b))

In the angular deflection test, the angles 0.50, 1.00, 1.50 and 2.00 degrees at the joint were tested. The testing time under each angle was 30 minutes with a maximum hydrostatic pressure of 7 psi for 10 minutes. The results of the test are summarized in Table 5-5. The relationship between angle of rotation and shear load at the joint is shown in Fig. 5-9. Shear force varied from 226 to 350 lbs at the joint. No water leak was observed at the joint during the total test period of 2 hrs.

5.3.3 METHOD C: Shear Load Test (Fig. 5-1 (c))

The joint was tested under shear loading according to Method C. The load was applied at load cell No. 1 on the pipe and was increased in steps of 500, 1000, 2000, 2500 and 3500 lbs. The test results are summarized in Table 5-6 and the total testing time was 2.5 hours. The Shear load at the joint vs. applied load and the deflection of the pipe at the loading point vs. applied load are shown in Fig. 5-10 and Fig. 5-11 respectively. The maximum shear load at the joint was 2899 lbs. and there was no water leak. The maximum deflection of the pipe occurred at the point of loading (Cross-section 1-1 (Fig. 5-1c)) of the pipe were -2.22% (Extension) and 2.96% (Compression) (based on the pipe diameter) in the horizontal and vertical directions respectively.

5.3.4 Summary

The composite bladder performed as designed. Total testing time for Method A, B, and C were 0.5 hour, 2 hours and 2.5 hours respectively. No leakage was observed at the tested joint for all the testing conditions. In the shear load test, the maximum shear load at the joint was 2899 lbs. and there was no water leak. The maximum deflection of the pipe occurred at the loading point at maximum shear load (based on the pipe diameter). The vertical and horizontal deflections at the loading point (Cross-section 1-1 (Fig. 5-1c)) of the pipe were -2.22% (Extension) and 2.96% (Compression) (based on pipe diameter) respectively.

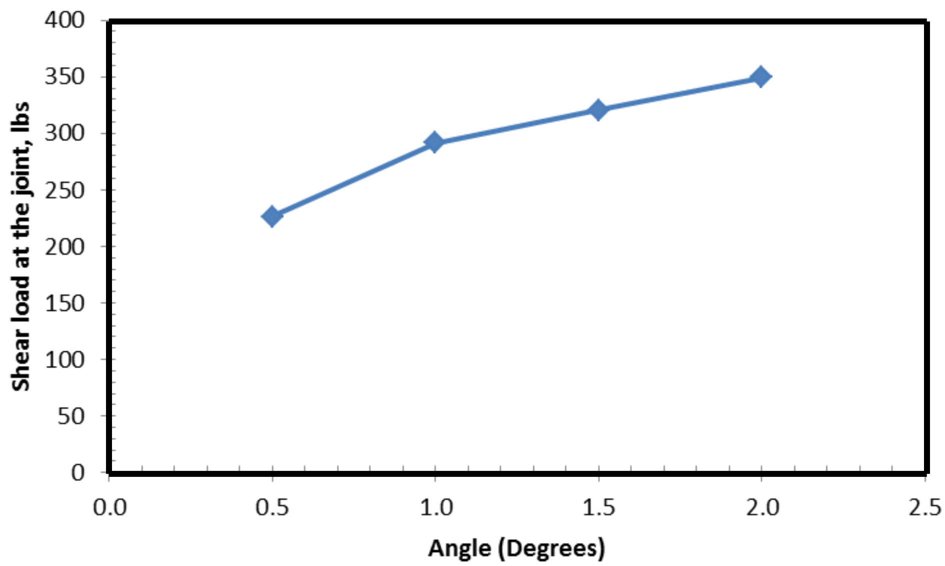


Figure 5-9: The Relationship between Angle and Shear Load in Test 3

Table 5-5: Results from Straight and Angular Deflection Tests (Test No. 3)

Method	Angle (^{o})	Pressure (psi)	Time (min)	Leakage	Remarks
A	0	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
B	0.5	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
	1.0	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
	1.5	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
	2.0	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
Remark	Up to 2^o	3 to 7 psi	Total 2.5 hrs	No Leak	No water leak, Bladder performed as designed

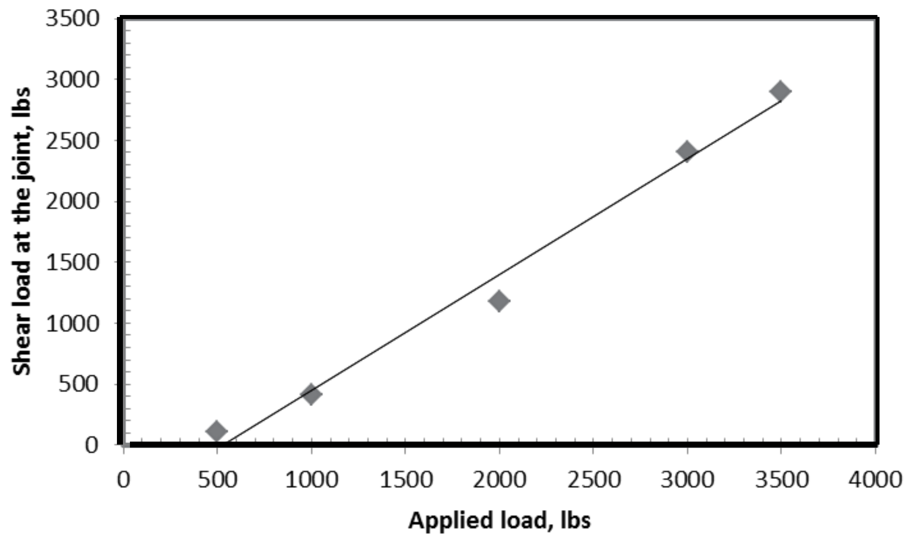


Figure 5-10: Applied Load vs. Shear Load in Test 3

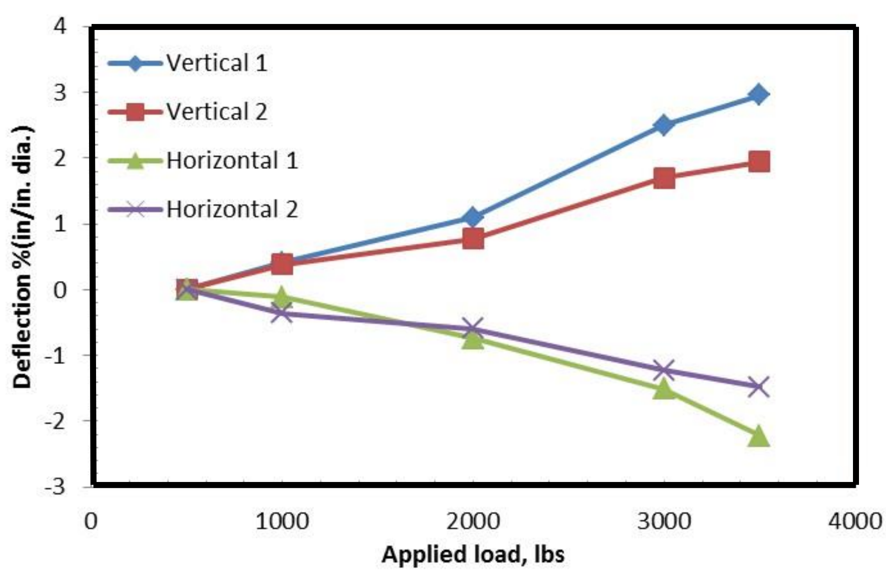


Figure 5-11: Applied Load vs. Deflection at the Loading Point during Test No. 3

Table 5-6: Results from Shear Load Test (Test No.3)

Intended Load (lb)	Pressure (psi)	Time (min)	Leakage	Actual Load Applied (lb)	Shear Load(lb)	Remarks
500	3	5	No	497	104	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
1000	3	5	No	994	412	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
2000	3	5	No	1815	1176	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
3000	3	5	No	2998	2410	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
3500	3	5	No	3495	2899	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
Remarks	Up to 7 psi	Total 2.5 hrs.	No leak	Maximum load 3495 lb.	Maximum shear 2899 lb	No water leak

5.4 Test No. 4. (PP – Double Wall Pipe)

5.4.1 METHOD A: Straight Pipe Joint Test (Fig. 5-1 (a))

The actual test setup is shown in Fig. 5-2(a). The test results are summarized in Table 5-7. No water leak was observed at the joint during the total test period of 30 minutes with maximum hydrostatic pressure of 7 psi for 10 minutes.

5.4.2 METHOD B: Angular Deflection Test (Fig. 5-1 (b))

In the angular deflection test, the angles 0.50, 1.00, 1.50 and 2.00 degrees at the joint were tested. The testing time under each angle was 30 minutes with a maximum hydrostatic pressure of 7 psi for 10 minutes. The results of the test are summarized in Table 5-7. The relationship between angle of rotation and shear load at the joint is shown in Fig. 5-12. Shear force varied from 197 to 350 lbs at the joint. No water leak was observed at the joint during the total test period of 2 hrs.

5.4.3 METHOD C: Shear Load Test (Fig. 1 (c))

The joint was tested under shear loading according to Method C. The load was applied at load cell No. 1 on the pipe and was increased in steps of 500, 1000, 2000, 2500 and 3500 lbs. The test results are summarized in Table 5-8 and the total testing time was 2.5 hours. The Shear load at the joint vs. applied load and the deflection of the pipe at the loading point vs. applied load are shown in Fig. 5-13 and Fig. 5-14 respectively. The maximum shear load at the joint was 2268 lbs. and there was no water leak. The maximum deflection of the pipe occurred at the point of loading (Cross-section 1-1 (Fig. 5-1c)) of the pipe were -2.16% (Extension) and 2.61% (Compression) (based on the pipe diameter) in the horizontal and vertical directions respectively.

5.4.4 Summary

The composite bladder performed as designed. Total testing time for Method A, B, and C were 0.5 hour, 2 hours and 2.5 hours respectively. No leakage was observed at the tested joint for all the testing conditions. In the shear load test, the maximum shear load at the joint was 2268 lbs. and there was no water leak. The maximum deflection of the pipe occurred at the loading point at maximum shear load (based on the pipe diameter). The vertical and horizontal deflections at the loading point (Cross-section 1-1 (Fig. 5-1c)) of the pipe were -2.16% (Extension) and 2.61% (Compression) (based on pipe diameter) respectively.

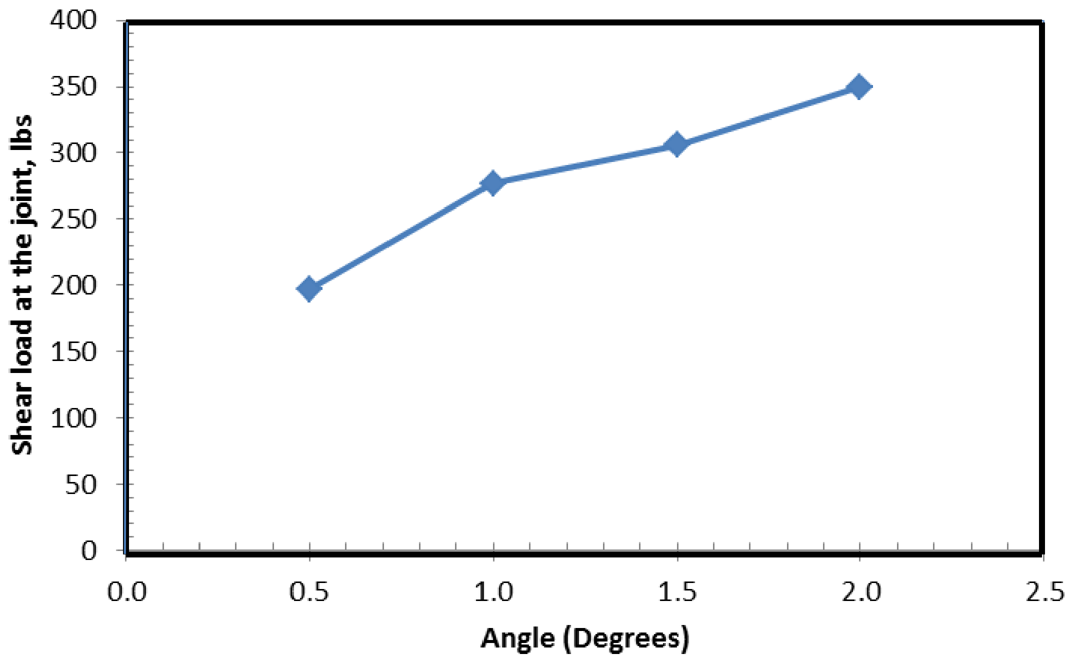


Figure 5-12: The Relationship between Angle and shear Load in Test 4

Table 5-7: Results from Straight and Angular Deflection Tests (Test No. 4)

Method	Angle (^{o})	Pressure (psi)	Time (min)	Leakage	Remarks
A	0	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
B	0.5	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
	1.0	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
	1.5	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
	2.0	3	5	No	Total testing time was 30 minutes. No leakage.
		4	5	No	
		5	5	No	
		6	5	No	
		7	10	No	
Remark	Up to 2^{o}	3 to 7 psi	Total 2.5 hrs	No Leak	No water leak, Bladder performed as designed

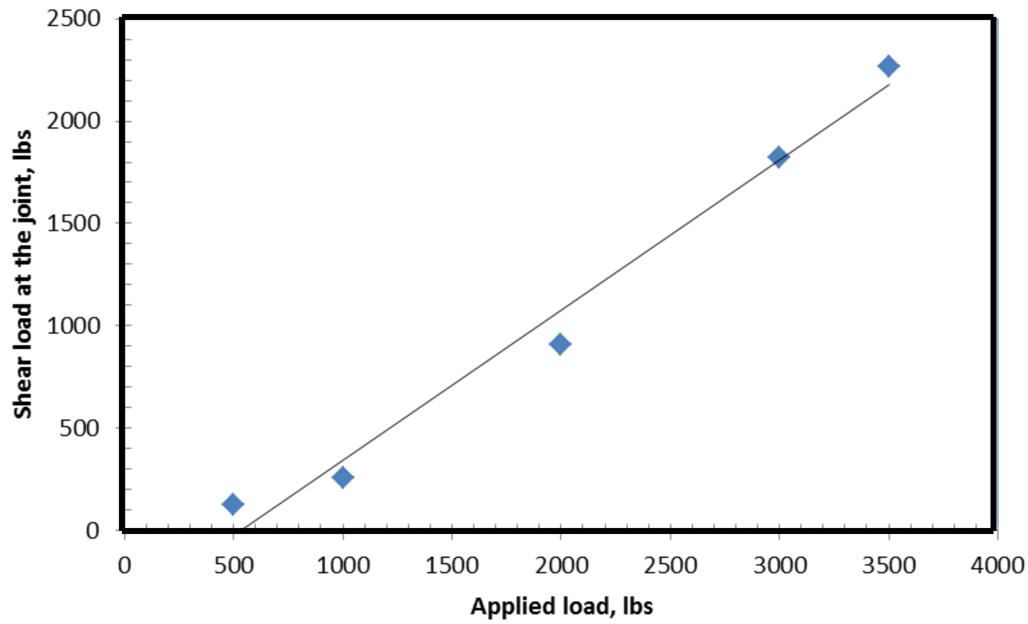


Figure 5-13: Applied Load vs. Shear Load in Test 4

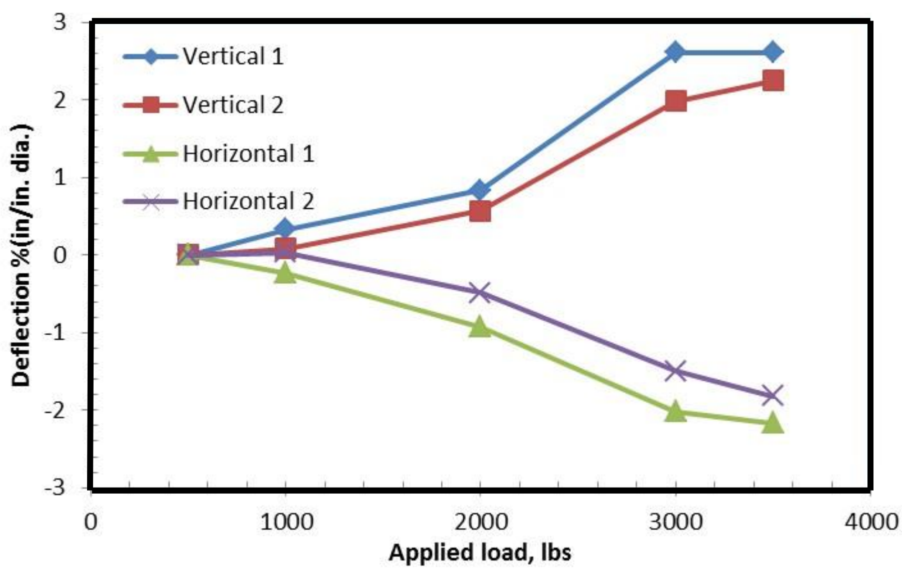


Figure 5-14: Applied Load vs. Deflection at the Loading Point during Test No. 4

Table 5-8: Results from Shear Load Test (Test No.4)

Intended Load (lb)	Pressure (psi)	Time (min)	Leakage	Actual Load Applied (lb)	Shear Load (lb)	Remarks
500	3	5	No	497	128	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
1000	3	5	No	994	260	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
2000	3	5	No	1815	908	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
3000	3	5	No	2998	1822	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
3500	3	5	No	3495	2268	Total test time was 30 minutes. No leak was observed.
	4	5	No			
	5	5	No			
	6	5	No			
	7	10	No			
Remarks	Up to 7 psi	Total 2.5 hrs.	No leak	Maximum load 3495 lb.	Maximum shear 2268 lb	No water leak

5.5 Summary

The testing of the 30-in diameter PP pipe-joints (Triple Wall and Double Wall) was performed at the CIGMAT Laboratory, University of Houston, Houston, Texas. Based on the four joints tested, following conclusions were discovered:

1. Straight Test: There was no leakage at the 30-in. PP Pipe-joints (Triple Wall and Double Wall). Each of the joint was tested without any external load for a total testing time of 30 minutes.
2. Shear Test: The Triple Wall joint was subjected to a maximum shear force of 2870 lb. (equivalent to 96 lb/in diameter) and there was no leakage. The Double Wall joint was subjected to a maximum shear force of 2899 lb. (equivalent to 97 lb/in diameter). The total testing time was 2.5 hours for each test.
3. Angular Test: During the angular test, the joint was subjected to a maximum rotation of 2^o at the joint. The total testing time was 2 hours and the shear load at the joint of the triple wall pipe varied from 292 to 533 lbs. during the angular test. On double wall pipe, the loading was 350 lbs for both joint tests. There was no leakage in all the tests.

CHAPTER 6

MANHOLE EVALUATION

Polyurethane and Polyuria coating materials are being used to both repair and maintain concrete and brickwork manhole structures. These coatings are used to prevent manhole structures from leaking and aging. The coating material on manholes was investigated to evaluate the performance in terms of overall condition, amount of water leakages, movement of coating and change in color. Full scale tests were performed in the laboratory.

6.1 Test Results and Discussions

6.1.1 Concrete Manhole

Concrete specimens used for the bonding test were first evaluated to quantify their quality. All the test specimens for the laboratory tests were prepared in the CIGMAT laboratory at the University of Houston.

6.1.1.1 Quality Control

To ensure the quality of the concrete specimens used in the coating studies the unit weight and pulse velocity of the specimens were measured.

6.1.1.1.1 Unit Weight and Pulse Velocity

Concrete: The unit weight of concrete specimen used for the bonding test was 25.5 kN/m^3. The pulse velocity had a mean value of 4748 m/s.

6.1.1.1.2 Strength

Concrete: The average flexural strength was 8.3 MPa (1200 psi).

6.1.1.2 Coating Materials

(a) Full-Scale Test

In order to evaluate the potential of applying the coating SPECTRASHIELD coating, manhole with wet surface (simulating in service condition) and water leaks was used. Performance of coating was evaluated for a period of three weeks.

Test Procedure

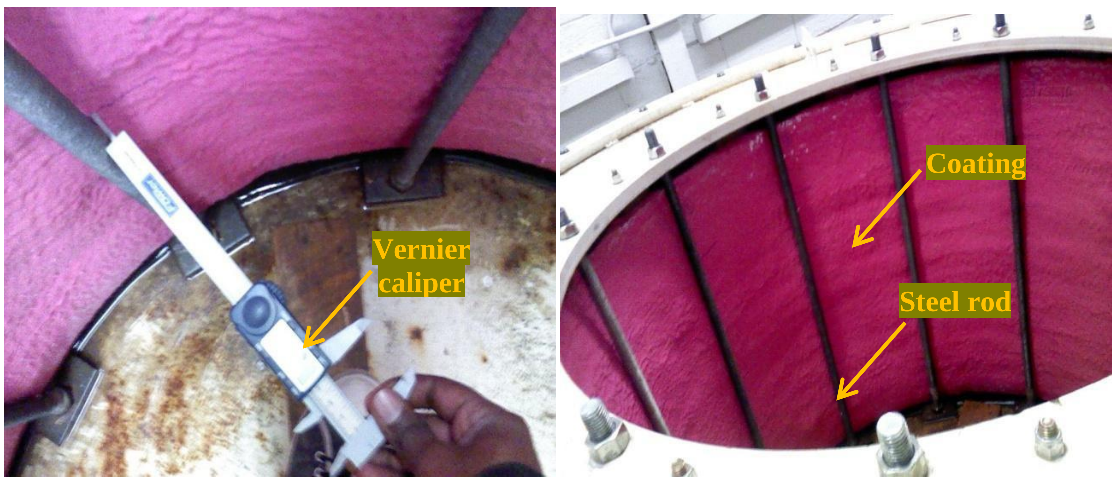
1. The concrete manhole was placed in a steel testing chamber. Water swelling agent was applied on the bottom and top ring of the concrete manhole to avoid leakage of water from the ends.
2. The outside chamber was filled with water.
3. Initial readings of coating with respect to the steel rods were taken using a vernier caliper.
4. The hydrostatic pressure was increased to 5 psi on the same day (Step (a)).
5. Since the concrete manhole coating was stable at 5 psi, the pressure was increased to 7.5 psi after one hour (Step (b)).
6. Once the pressure was increased to 7.5 psi, the spacing between the steel rod and coating were noted.
7. After 17 hours, the pressure was dropped to 7 psi. The coating distance from the rod was measured.
8. The hydrostatic pressure was increased to 9 psi after taking the readings (Step (c)). (Fig. 6-1).
9. There was no pressure drop in 24 hours. The spacing between the steel rods and coatings were measured.
10. The hydrostatic pressure was increased to 11 psi (Step (d)).

11. After 4 days, the pressure dropped to 5 psi. The spacing between the steel rods and coatings were measured (Step (e)). (Fig. 6-2(a)).

12. The testing was continued at hydrostatic pressure of 2 psi. The spacing between the steel rods and coatings were measured (Step (f)).

13. The pressure remains at 2 psi for four days (Step (h &i)). (Fig. 6-2(b)).

14. The hydrostatic pressure was reduced to zero and the coating movement was monitored (Step (j)).



(a) Vernier caliper for deformation **(b) View of the coating measurement**

Figure 6-1: Coating at Hydrostatic Pressure of 9 psi

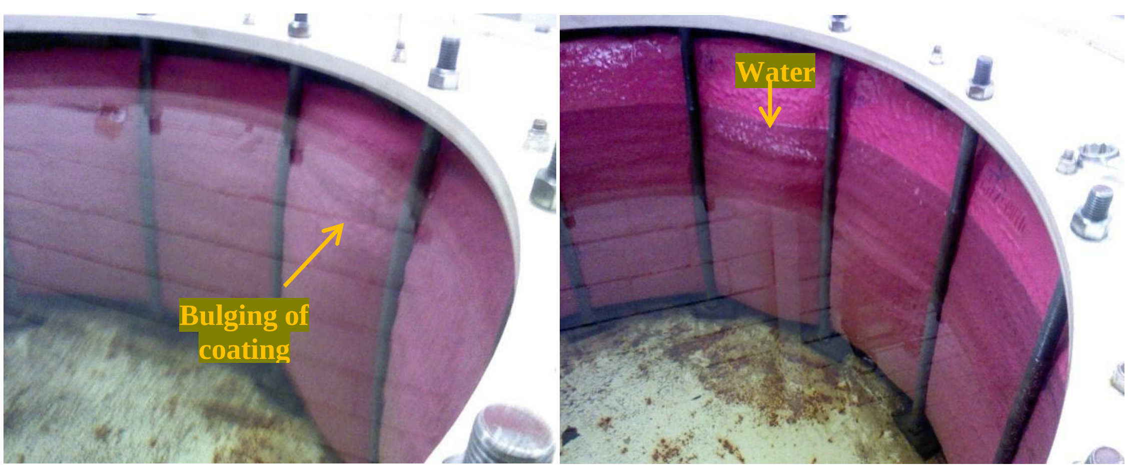


Figure 6-2: Coating at Pressure (a) 11 psi (b) 2 psi (after 17 days)

(b) Pressure Test

The hydrostatic pressure history used to test the manhole is shown in Fig. 6-3. The pressure was initially increased in steps up to 11 psi and then reduced. The total test period was 20 days.

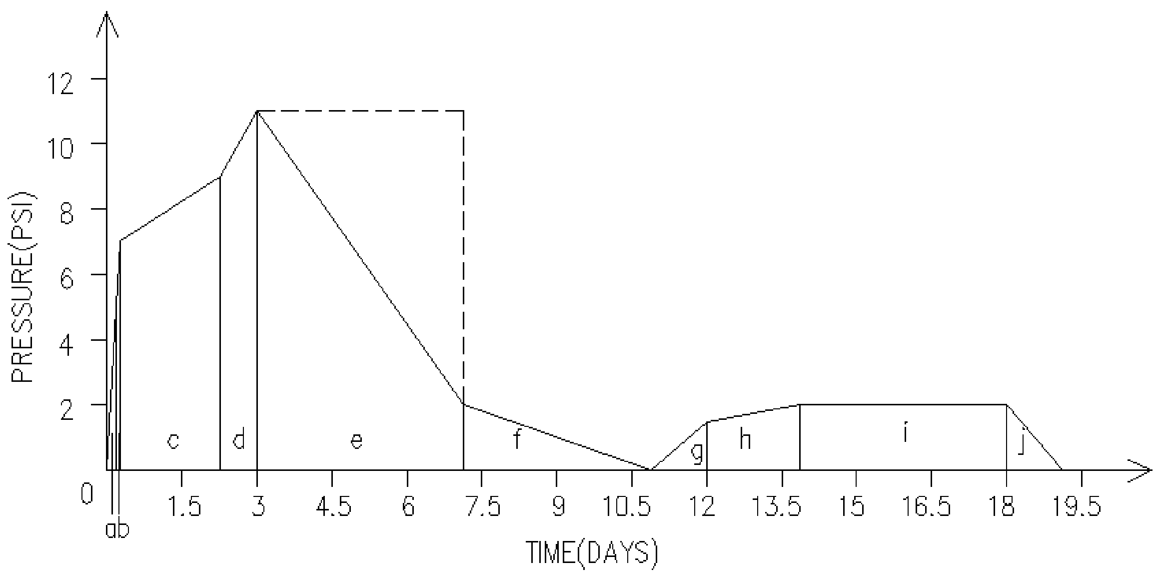


Figure 6-3: Applied Hydrostatic Pressure Versus Time

Visual Inspection: Deformation of coating at each hydrostatic pressure and leakages on the coated concrete surface were visually monitored and inspected regularly.

6.1.1.2.1 Application

No immediate defects (blistering, cracking, discoloration, spalling, sticking to the finger after 48 hours of application) were observed on the coated surfaces.

6.1.1.2.2 Performance

The coating was tested under a hydrostatic pressure of up to 11 psi over a period of three weeks. For monitoring purposes the coating movement was measured in reference to the twelve steel rods. The coating was inspected on a regular basis to identify any visible defects.

Each section was evaluated for (i) Overall condition (ii) Amount of water leakages (iii) Movement of coating, and (iv) Change in color.

Based on the testing for a period of 21 days, the following observations were observed

(i) The overall condition was good

(ii) No water leaked through the coating

(iii) The movement of the coating was observed with an increase in pressure, partly due to debonding of the coating from the concrete manhole.

(iv) No color change.

6.1.1.2.3 Bonding Strength

Wet concrete specimens were coated to simulate the Full-scale testing conditions. Total of 3 bonding tests with concrete were performed. The bonding strength varied from 106 to 160 psi (average 139 psi). All the failures (100%) were by bonding (type III). Debonding could be one of the reasons for the movement of the coating during the pressure test.

6.1.2 Brick Manhole

6.1.2.1 Coating Materials

(a) Full-Scale Test

In order to evaluate the potential of applying the coating SPECTRASHIELD coating, manhole with wet surface (simulating in service condition) and water leaks was used. Performance of coating was evaluated for a period of three weeks.

Test Procedure

1. The brick manhole was placed in a steel testing chamber. Water swelling agent was applied on the bottom and top ring of the brick manhole to avoid leakage of water from the ends.
2. The outside chamber was filled with water.
3. Initial readings of coating with respect to the steel rods were taken using a vernier caliper.

4. The hydrostatic pressure was increased to 6.5 psi on the same day (Step (a)).

5. After 24 hours, the pressure dropped to 5 psi. The spacing between the steel rods and coatings were measured (Step (b)) (Fig. 6-4).

6. Once the pressure was increased to 7.5 psi, the spacing between the steel rod and coating were noted (Step (c)) (Figure 7(a)).

7. After half an hour, the hydrostatic pressure was reduced to zero, the spacing between the steel rod and coating were noted and the coating movement was monitored (Step (d)).

(a) Vernier Caliper for Deformation (b) View of the Coating Measurement

Figure 6-4: Coating at Hydrostatic Pressure of 6.5 psi

Figure 6-5: Coating at Pressure (a) 7.5 psi (b) 0 psi (after 2 days)

(b) Pressure Test

The hydrostatic pressure history used to test the manhole is shown in Fig.6-6. The pressure was initially increased in steps up to 7.5 psi and then reduced. The total test period was 2 days.

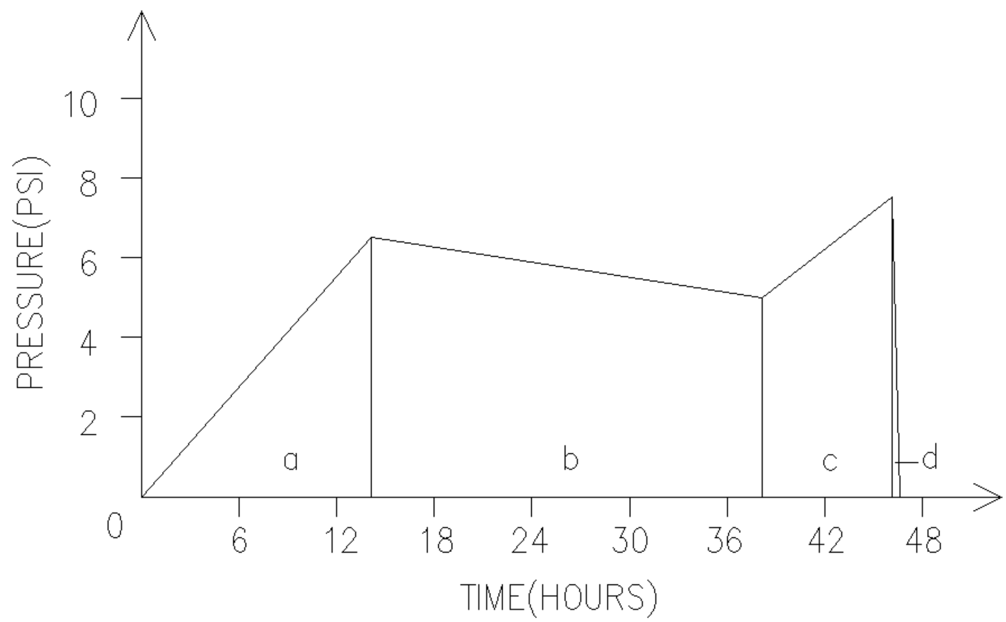


Figure 6-6: Applied Hydrostatic Pressure Versus Time

Visual Inspection: Deformation of coating at each hydrostatic pressure and leakages on the coated brick surface were visually monitored & inspected regularly.

6.1.2.1.1 Application

The coating was applied successfully under wet conditions in the CIGMAT laboratory. Coating was applied with ease. Coating was inspected during and immediately after application. No immediate defects (blistering, cracking, discoloration, spalling, sticking to the finger after 48 hours of application) were observed on the coated surfaces.

6.1.2.1.2 Performance

The coating was tested under a hydrostatic pressure of up to 7.5 psi over a period of three weeks.

For monitoring purposes the coating movement was measured in reference to the twelve steel rods. The coating was inspected on a regular basis to identify any visible defects. Each section was evaluated for (i) Overall condition (ii) Amount of water leakages (iii) Movement of coating, and (iv) Change in color.

Based on the testing for a period of 2 days, following observations were observed

(i) The overall condition was good.

(ii) No water leaked through the coating

(iii) The movement of the coating was observed with an increase in pressure, partly due to debonding of the coating from the brick manhole.

(iv) No color change.

6.2 Summary

Based on the full-scale test and the bonding test, the following observations were made:

1. At hydrostatic pressure above 6.5 psi and 11 psi, notable bulging was observed for brickwork and concrete manhole respectively.
2. Reducing the hydrostatic pressure recovery of the coating to near original condition.
3. No water leak was noticed through the coating.
4. Overall condition of the manhole was good.
5. Movement of the coating was observed with increase in pressure partly due to debonding of the coating from the concrete and brick manhole.
6. No color change of coating material was observed.
7. The movement of the coating was uneven with depth and was influenced by the applied pressure.

CHAPTER 7

MODELING OF POLYPROPYLENE PIPE

7.1 Introduction

In this chapter, a finite element model of polypropylene pipe was developed to analyze the polypropylene pipe under loads applied in shear load test. The FEM results were compared with the actual experimental results.

7.2 Mechanical Properties

Polypropylene pipe has good mechanical properties when compared with other plastic pipes. The mechanical properties of polypropylene pipe used in this model are listed in Table 7-1.

Table 0-1: Mechanical properties of polypropylene pipe (George fischer piping systems, 2010)

Properties		Unit	Polypropylene Natural PP-R
Density		lb/cu.in (pcf)	0.0325(56.2)
Tensile Strength@73°F properties	Strength	psi	3625
	Specific Strength	N-ft/N	9,300
Modulus of Elasticity@73°F	Strength	psi	130,500
	Specific Modulus	N-ft/N	334,615
Linear Thermal Expansion		in/in/˚F	$0.5x10^{-4}$
Poisson's ratio			0.41

7.3 Finite Element Model of Polypropylene Pipe

Polypropylene pipe of diameter 30 inch and length of 6 feet 3 inch was modeled using finite element method. Different mesh element sizes were used according to the stress concentration. Polypropylene pipe was modeled and analyzed without any joint in order to investigate the effect of joint under loading. Mesh details are shown in the Table 7-1. The 3D modeling polypropylene pipes are shown in Fig. 7-1 and 7-2.

Table 7-2: Finite element mesh information

Mesh type	Triangular
Mesher Used:	Curvature based mesh
Jacobian points	4 Points
Maximum element size	0.43 in
Minimum element size	0.17 in
Total Nodes	79660
Total Elements	44906

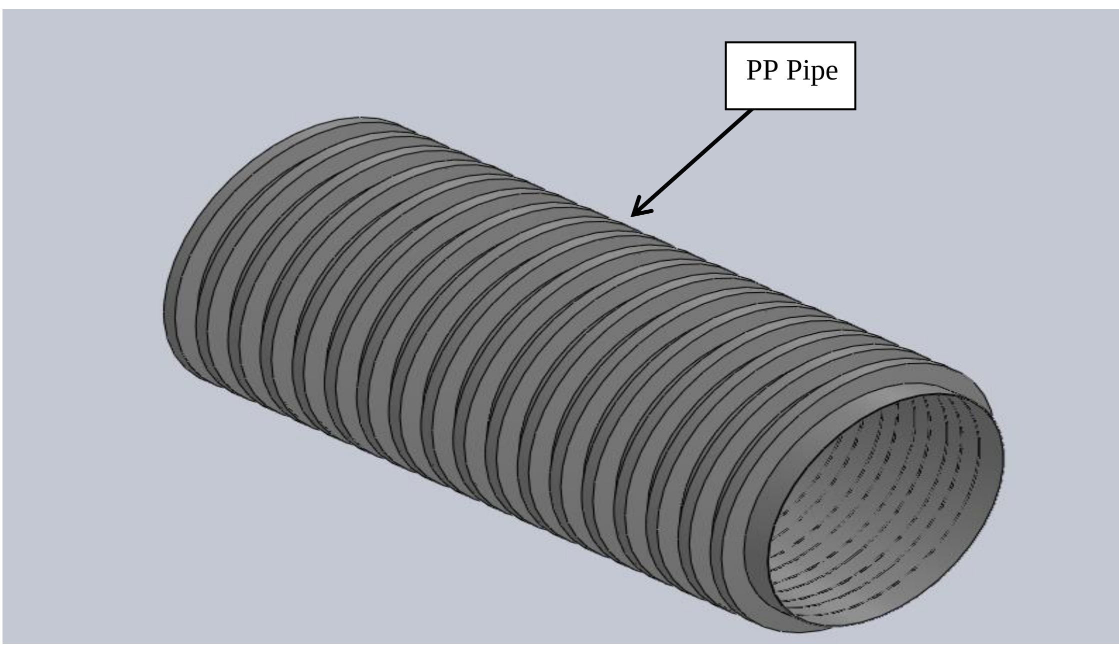


Figure 7-1: 3D- Modeling of Polypropylene Pipe Without Joint

Table 7-3: Finite element mesh information

Mesh type	Triangular
Mesher Used:	Curvature based mesh
Jacobian points	4 Points
Maximum element size	0.38 in
Minimum element size	0.076 in
Total Nodes	2036926
Total Elements	1035280

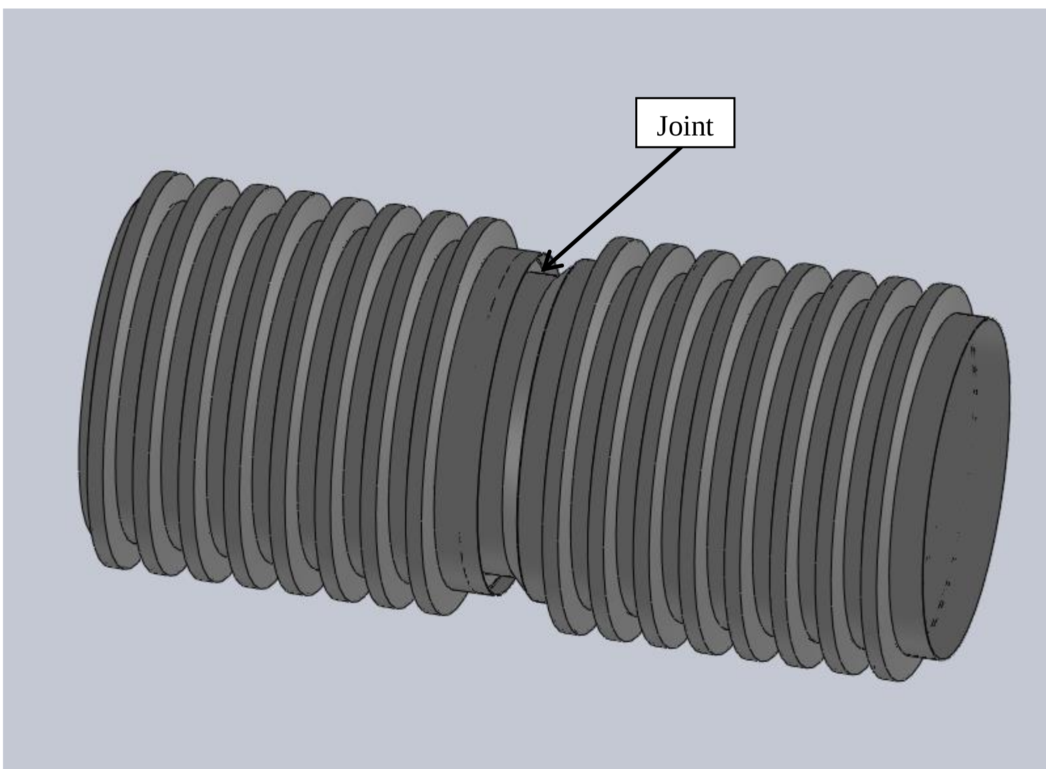


Figure 7-2: 3D- Modeling of Polypropylene Pipe with Joint

7.3.1 Shear Load Test

In this test, the load was applied at point 1 as shown in Fig. 5-1 (C) and was increased from 500 lbs to 3500 lbs. The deformation was determined at the location of the joint from the result. From the finite element method, the maximum stress corresponding to the applied load of 3500 lbs was calculated. The distance between two supports was 4 feet 3 inch. The load was applied at 2 feet 3 inch from the right corner of the polypropylene pipe as shown in the Fig. 7-3.

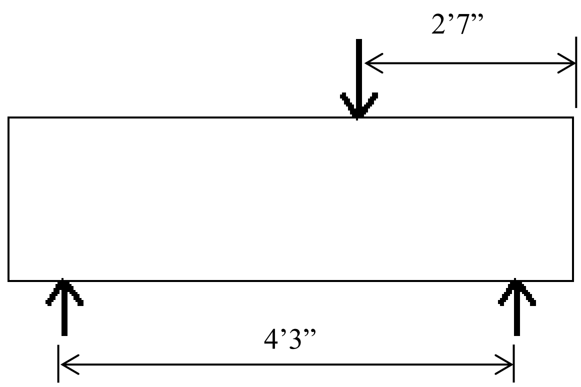


Figure 7-3: Testing Set up of Polypropylene Pipe

Without Joint Case

The second deviatoric stress invariant distribution and the deformed shape of polypropylene pipes without joint case are shown in Fig. 7-4 and 7-5. From the second deviatoric stress invariant distribution and deformed shape of the pipe, it is clear that due to the absence of the joints, the stress was uniformly distributed and no weak spot were formed. Also deformation at the location of joint was low when compared with the deformation with joint case.

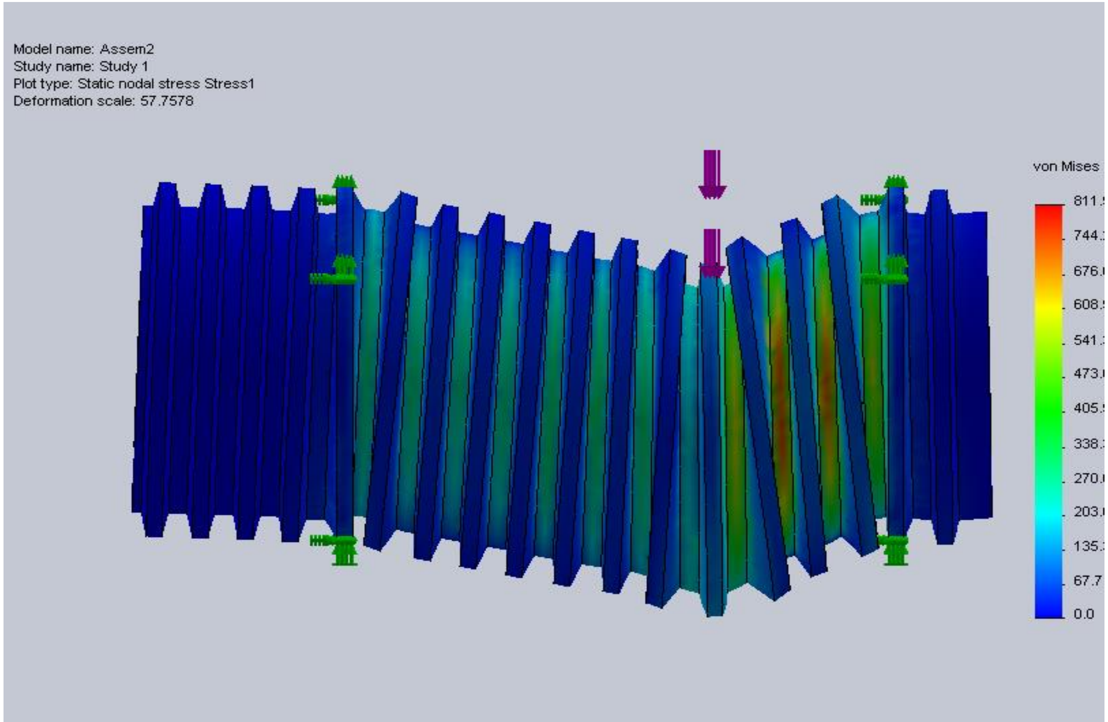


Figure 7-4: The second deviatoric stress invariant Distribution of Polypropylene Pipe Without Joint

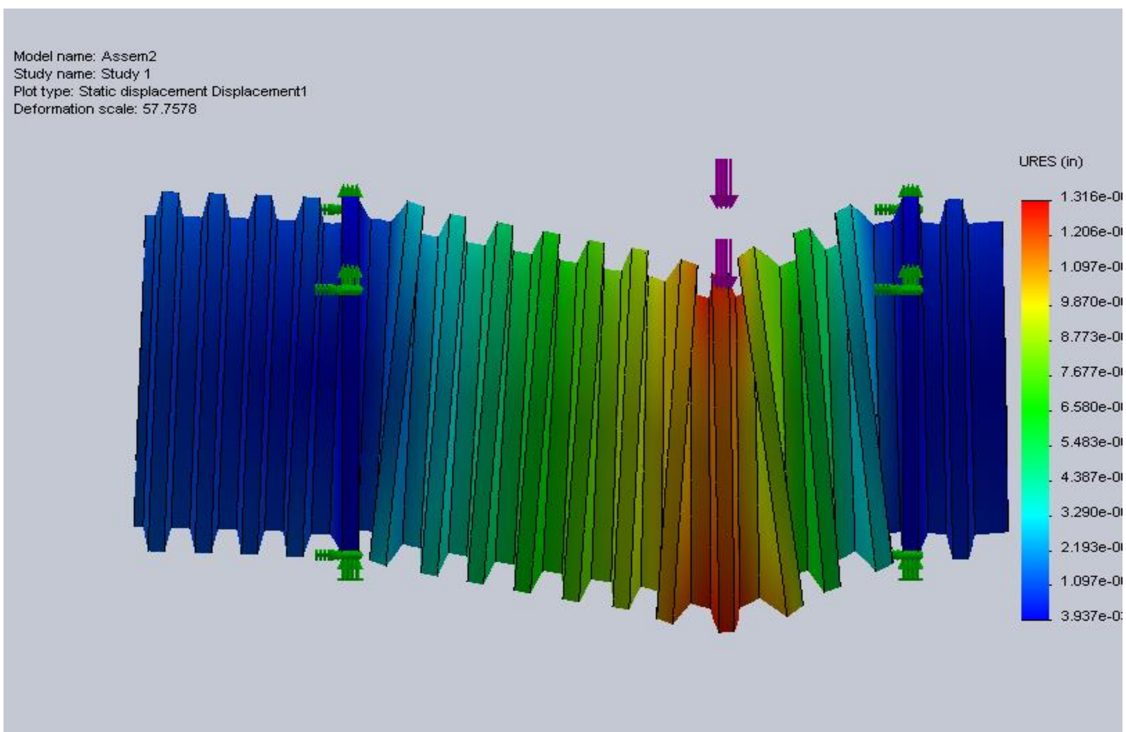


Figure 7-5: Deformed Polypropylene Pipe without Joint Under the Applied Loading

With Joint Case

For with joint case, the second deviatoric stress invariant distribution and the deformed shape of polypropylene pipes are shown in the Fig. 7-6 and 7-7. The stress distribution and displacement graph at the location of joint are shown in the Fig. 7-8 and 7-9. From these graphs, it is obvious that the stress concentration was taken place at joint rather than other parts of the polypropylene pipe. Also, from the Fig. 7-9, excess deformation was occurred at the joint of the polypropylene pipe.

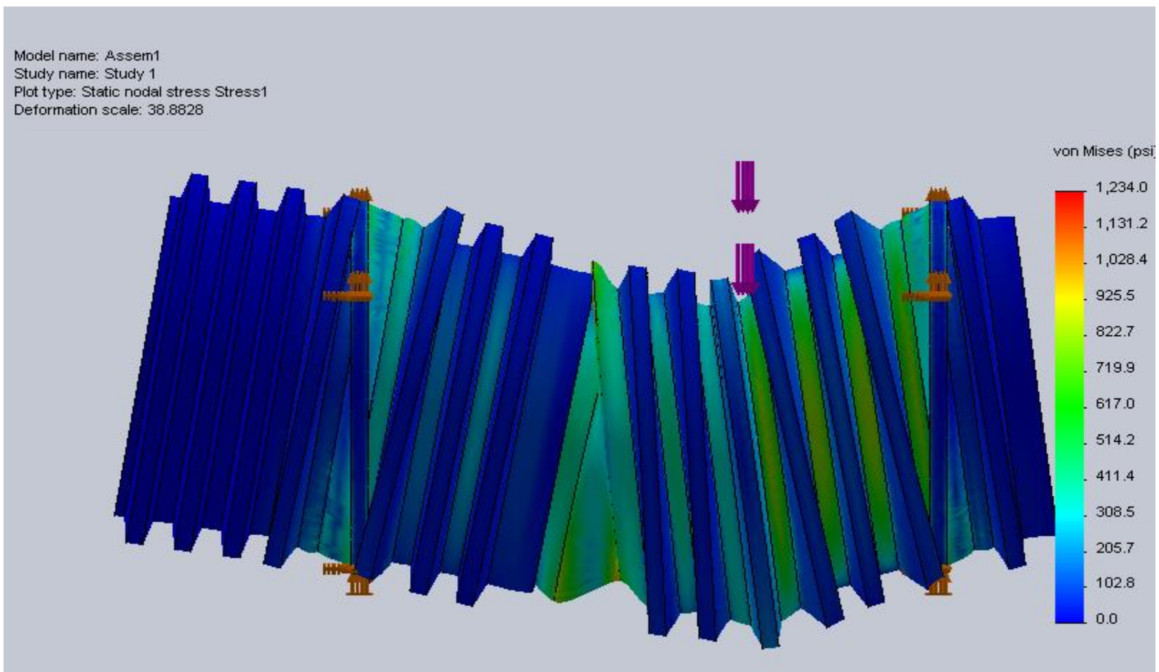


Figure 7-6: The second deviatoric stress invariant Distribution of Polypropylene Pipe with Joint Under the Applied Loading

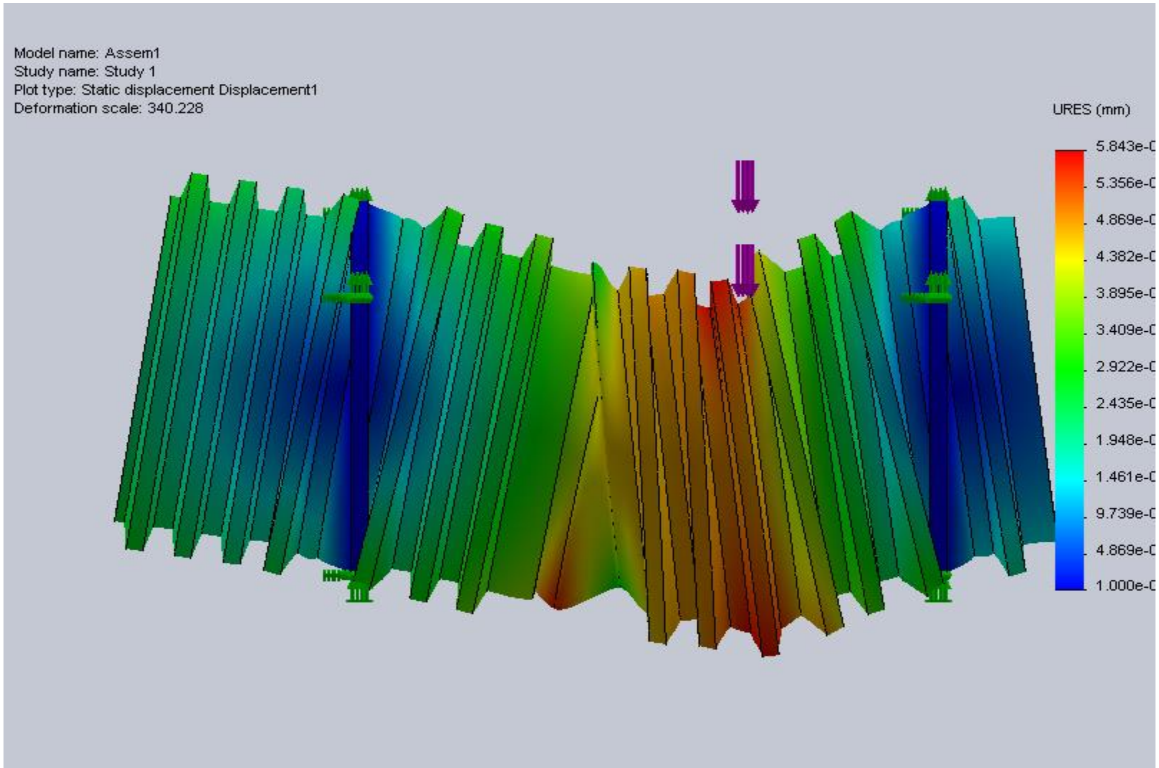


Figure 7-7: Deformed Polypropylene Pipe with Joint Under the Applied Loading

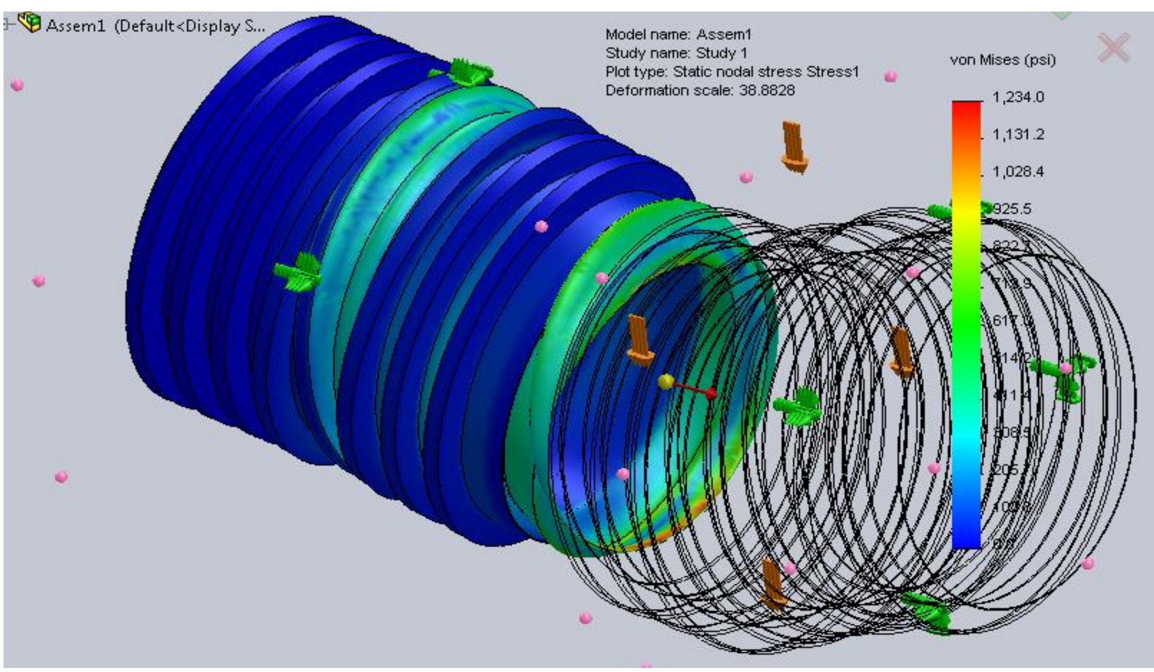


Figure 7-8: Cut Section of Joint Stress Distribution

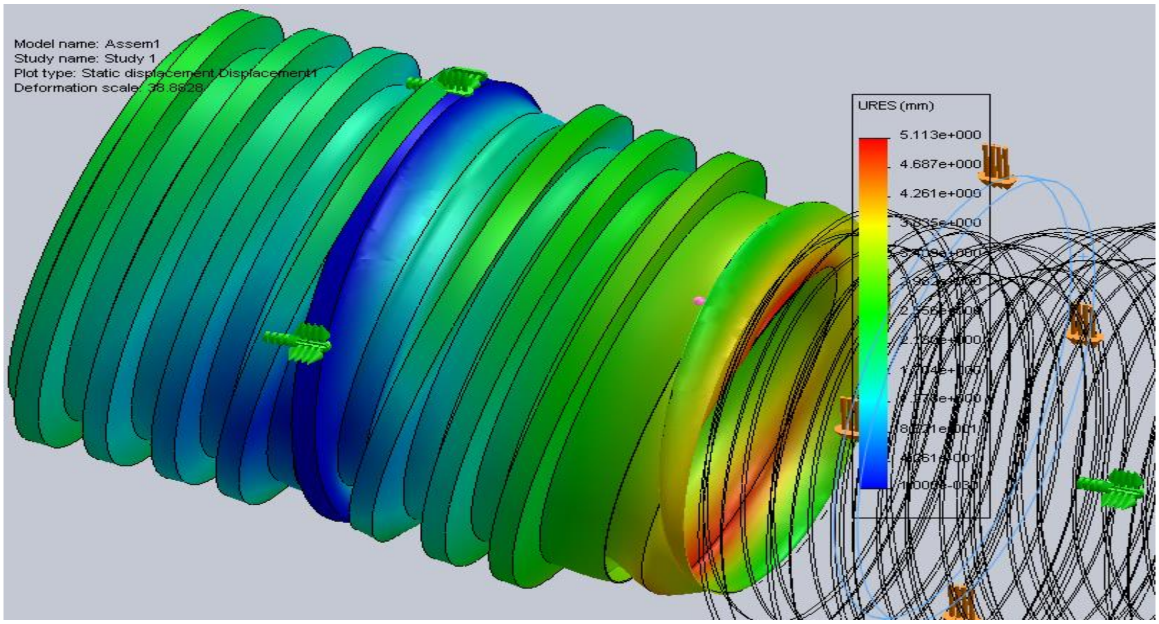


Figure 7-9: Cut section of Joint Displacement Graph

7.3.1.1 Discussion of Results

The maximum percentage of deformation of the pipe occurred at the location of joint with and without joint conditions were 2.2% and 0.13%. Deflections at joints from numerical model are compared with experimental results for each loading. Also the same deflections are compared with polypropylene pipe without joints which is plotted in the Fig. 7-10. This graph showed good correlation between experimental and numerical deflection values.

The maximum the second deviatoric stress invariant developed at the location of the joint with and without joint cases are 1234 psi and 473 psi respectively under the loading of 3500 lbs. The stress is high at the point of loading as well as at the joints. At the top and bottom of the joints, the nature of stresses are compression and tensile respectively as shown in the Fig. 7-6.

Hence it is observed from the results that the stresses due to external loading tend to get concentrated at the joints. The trend of experimental result matches considerably with finite element analysis result.

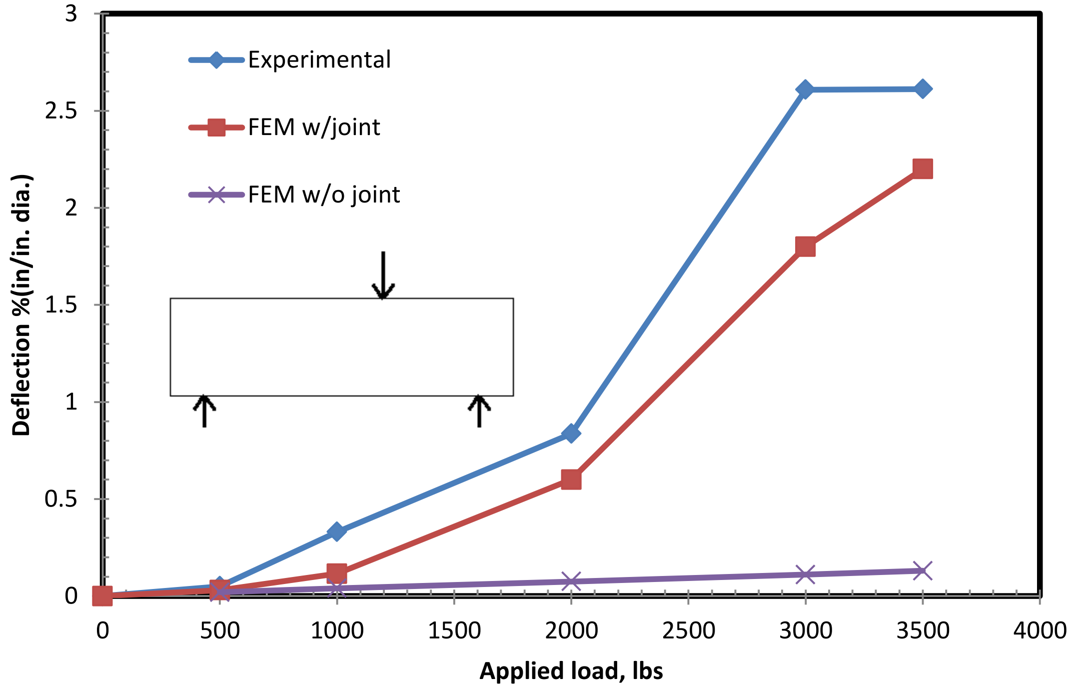


Figure 7-10: Deflection Percentage at Joints

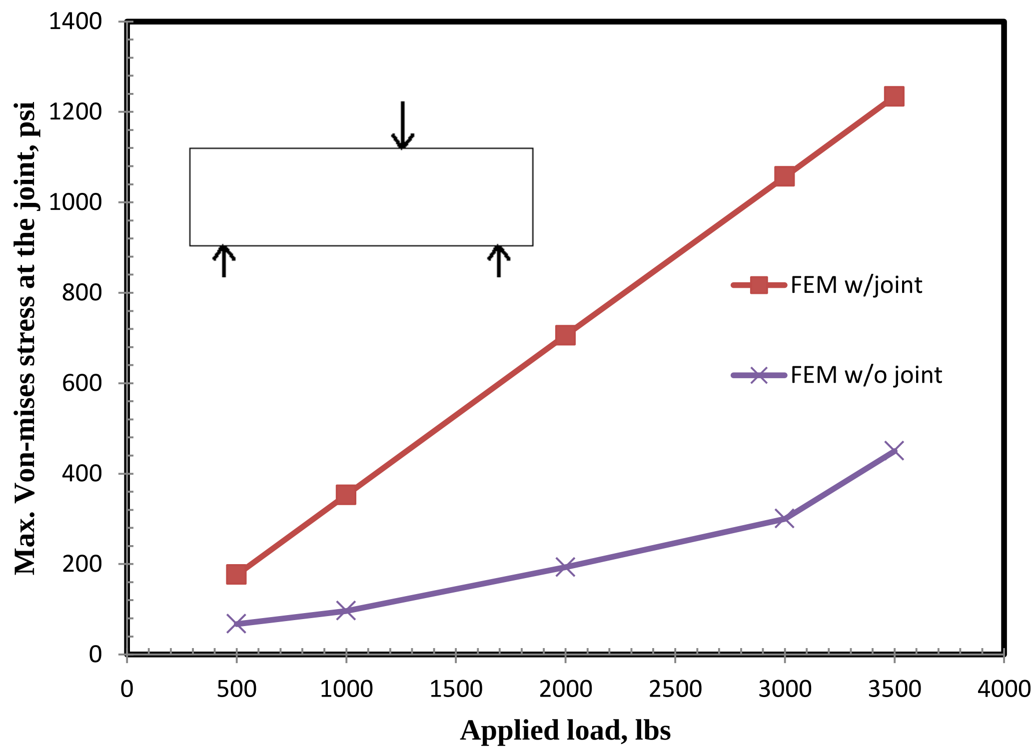


Figure 7-11: The second deviatoric stress invariant at the Joint During Shear Load Test

Effect of Elastic Modulus

Polypropylene pipe without any joint are analyzed by varying the values of elastic modulus. Percentages of deflection are varied considerably as the values of elastic modulus changes. With the decrement of elastic modulus, the values of deflection are getting increased which is shown in the Fig. 7-12.

Effect of Poisson's Ratio

Polypropylene pipe without any joint are analyzed by varying the values of poisson's ratio. Percentages of deflection are not that much sensitive with the variation of poisson's ratio value when compared with the previous case. With the decrement of poisson's ratio values, the values of deflection are getting decreased which is shown in the Fig. 7-13.

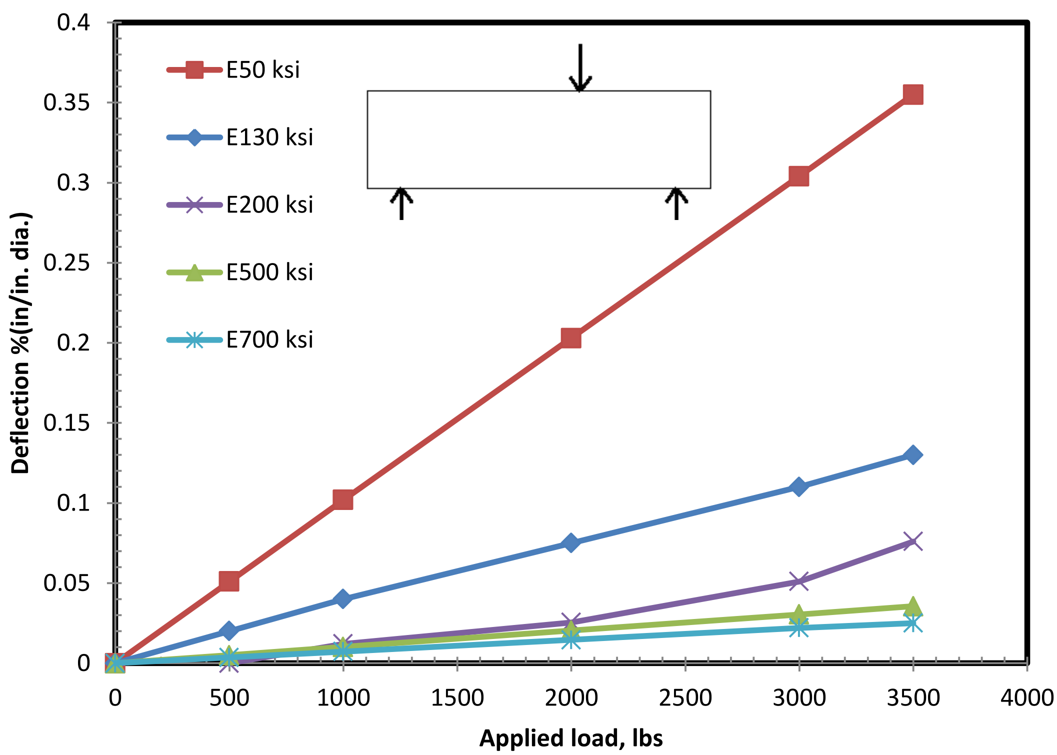


Figure 7-12: Effect of Elastic Modulus on Deflection for Without Joint Case

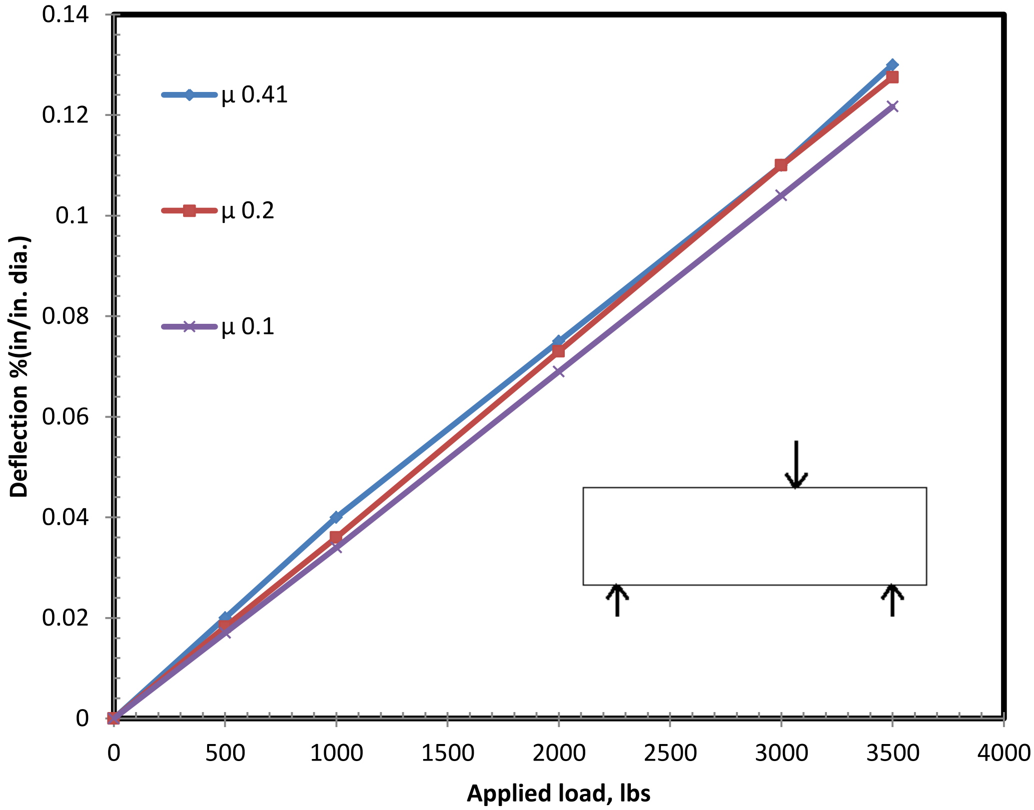


Figure 7-13: Effect of Poisson's Ratio on Deflection for Without Joint Case

7.3.2 Angular Test

In the Angular test numerical analysis, one end of the polypropylene pipe was moved against the joint with the predetermined angles which are 0.5˚, 1˚, 1.5˚ and 2˚. Two sets of analysis were done with and without the joints. The results were compared to investigate the effect of joints in terms of resistance to angular movement of the pipe.

The second deviatoric stress invariants induced due to the 2˚ angular movement with and without joints are shown in the Fig. 7-15 and 7-14 respectively. With absence of joints, the stresses were uniformly distributed throughout the left side of the pipe. Whereas due to the presence of joints in the Fig. 7-15, the stresses were only concentrated on the joint of polypropylene pipe.

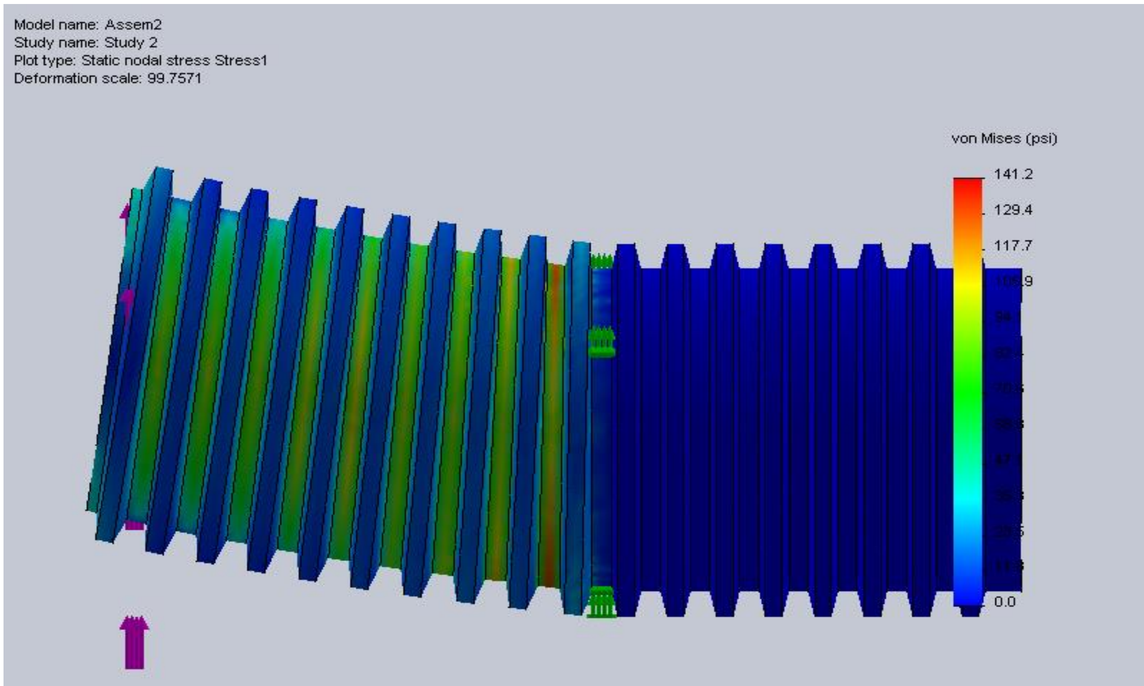


Figure 7-14: The second deviatoric stress invariant Distribution due to Angular Movement (Without Joint Case)

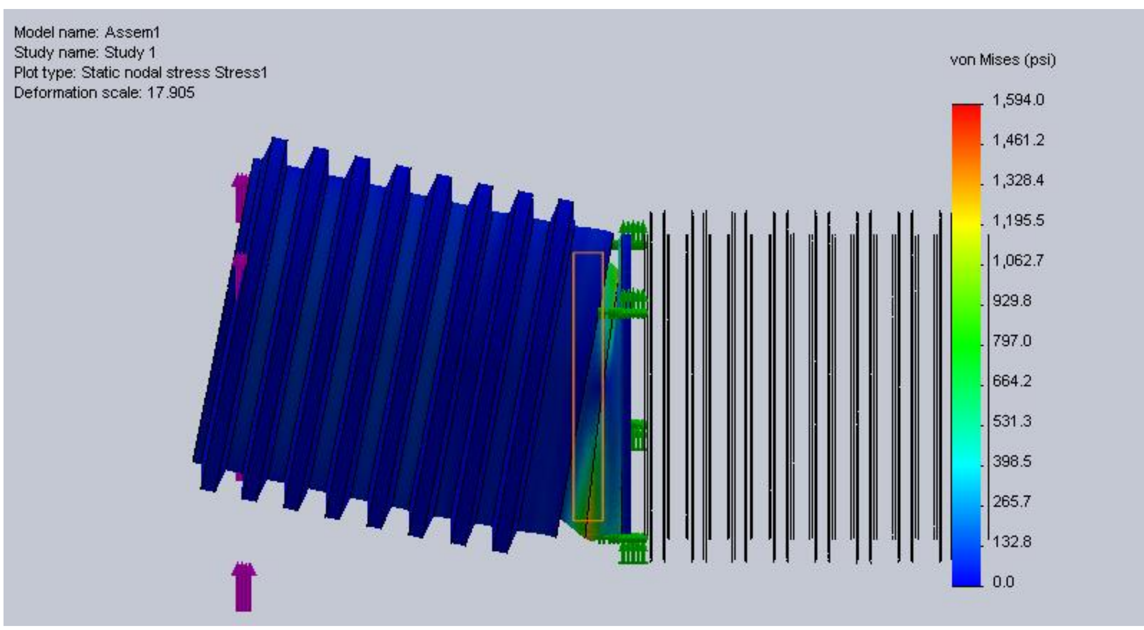


Figure 7-15: The second deviatoric stress invariant Distribution due to Angular Movement (With Joint Case)

7.3.2.1 Discussion of Results

The second deviatoric stress invariant developed are plotted in Fig. 7-16 for with and without joint cases. The maximum the second deviatoric stress invariant at the location of joint with and without joint cases were 1594 psi and 141 psi respectively. Results indicated that development of stress is very high at the joint because of the weak portion. However the stress was taken by each portion of the polypropylene pipe because of the absence of joints.

Hence it is clear that the presence of joints would act as a weak spot of the pipe where the extra stresses will get concentrated. Though the stresses were comparatively high at the joint, it is still within the yield limit of the polypropylene pipe.

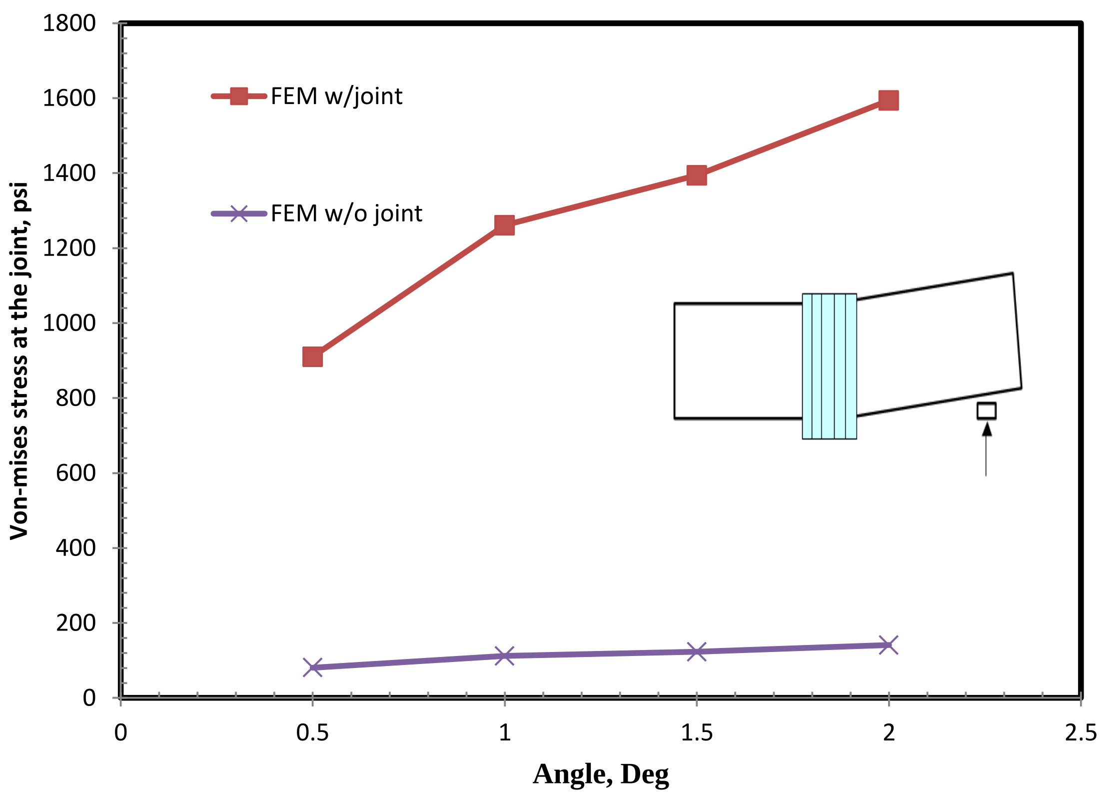


Figure 7-16: The second deviatoric stress invariantes at the Joint During Angular Test

7.4 Summary

Based on the above analysis, the following conclusions were discerned:

1. The maximum deflection occurred at the joint is 2.2% for the applied load of 3500 lbs. With the absence of joint the value of maximum deflection was reduced to 0.09%. The maximum deflection value from the experiment was 2.61%.
2. The maximum von-mises at the joint was 1234 psi for the applied load of 3500 lbs. For without joint condition, the maximum the second deviatoric stress invariant developed was 473 psi.
3. In the case of angular test analysis, the maximum the second deviatoric stress invariant developed at the joint with and without joint cases are 1594 psi and 141 psi respectively for 2˚ angular movement.
4. In the above analysis, the stresses were uniformly distributed throughout the pipe with absence of joint. However with joint condition, the stress was concentrated only at the joint because of the weak spot.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

This study focused on the curing (Chemi-resistivity) and piezo-resistive behavior of oil well cement and Portland cement with the addition of various additives. Pipe joint connection of polypropylene was evaluated using straight pipe test, angular and shear load test. Effectiveness of coating on concrete and brickwork manholes was studied. Based on the study the following conclusions can be determined:

1. With the addition of 0.075% carbon fiber, the resistivity of the cement slurry decreased from about 1000 Ω-m to 2000 Ω-m. In the modified cement the resistivity increased by 400 to 500% times the initial resistivity after the cement slurry completely hardened as compared to the unmodified cement where the change in resistivity was minimum. Using vicat test, the initial and final setting time were found to be 6 hours and 7 hours respectively.

2. Addition of potassium silicate decreased the resistivity of hardened cement. Addition of 1% and 5% of potassium silicate reduced the resistivity by 40% and 10% respectively.

3. After setting of cement slurry, the resistivity of 5% foamed cement increased 11 times the initial resistance. For 1% foamed cement, the resistivity increased 35 times the initial resistivity, a considerable change was observed for hardening of oil well cement.

4. For Portland cement, the value of resistivity at the initial stage of hydration was considerably low with carbon fiber of 0.075% while the value of resistivity with 0% carbon fiber was

similar to oil well cement. This was the direct effect of function of carbon fiber on resistivity value of cement slurry. After the cement was hardened the increment of resistivity was low when compared with the oil well cement. Hence it could be ascertained that presence of 2 to 4% gypsum affected the conductive nature of cement slurry considerably. However the resistivity value of 0% carbon content Portland cement was similar to the corresponding oil well cement slurry as it was varied from 800 Ω-m to 1200 Ω-m.

5. Small stresses (comp.) can be sensed with the fiber content of 0.075%. The percentage change of strain at failure was around 0.2%. Whereas the compressive failure stress of around 20 MPa was sensed with the change of resistivity of 80% which was a good indication. Electrical resistivity was proven as a better indicator than strain.

6. Straight Test: There was no leakage at the 30-in. PP Pipe-joints (Triple Wall and Double Wall). Each of the joint was tested without any external load for a total testing time of 30 minutes.

7. Shear Test: The Triple Wall joint was subjected to a maximum shear force of 2870 lb. (equivalent to 96 lb/in diameter) and there was no leakage. The Double Wall joint was subjected to a maximum shear force of 2899 lb. (equivalent to 97 lb/in diameter). The total testing time was 2.5 hours for each test.

8. Angular Test: During the angular test, the joint was subjected to a maximum rotation of 2º at the joint. The total testing time was 2 hours and the shear load at the joint of the triple wall pipe varied from 292 to 533 lbs. during the angular test. On double wall pipe, the loading was 350 lbs for both joint tests. There was no leakage in all the tests.

9. At hydrostatic pressure above 6.5 psi and 11 psi, notable bulging was observed for brickwork and concrete manhole respectively.

10. Reducing the hydrostatic pressure recovery of the coating to near original condition.

11. No water leak was observed through the coating up to hydrostatic pressure 11 psi condition. No bulging of coating was observed up to 6.5 psi.

8.2 Recommendations

The following suggestions are recommended for future work:

1. Chemi-resistive behavior of oil well cement slurry was observed under air curing. Though, the chemi-resistive behavior was good under air curing, the same study could be done by changing different curing conditions such as water curing. Also the temperature and properties of water could be altered and evaluate the difference in the chemi-resistive behavior of oil well cement during setting. Also during air curing, the room temperature could be changed and study the effect of different room temperatures.

2. The cement would release heat energy during hydration. The setting time of cement could be correlated to the amount of heat energy evolved during setting of cement. And another correlation could be formed between heat energy released and resistivity during setting.

3. The physical properties of carbon fiber could be further modified. It might lead to a way for further decrement of amount of fiber used for chemi-resistive and piezo resistive behavior of

cement. Also there are chances that it might help to improve early strength development of cement slurry. So further studies could be done in these aspects.

4. The shape of the cement slurry specimen could be altered. Instead of cylindrical specimen, cubical specimen could be made and changes in chemi-resistive behavior could be studied for each shape of the specimen.

5. The amount of load applied on the polypropylene pipe during shear load test could be increased so that the ultimate strength of the polypropylene pipe could be found.

6. The testing of manhole was done under room temperature (23±2˚C) and evaluated the coating material behavior under different hydrostatic pressure. The same test could be done under different temperature and the difference in behavior of coating could be studied under each temperature condition. Because in the real time, the manhole with the coating material could be installed anywhere in the USA where the environmental temperature could vary considerably from place to place.

7. The possible failure of coating material on manhole would be failure of bonding between coating material and concrete/brickwork. The surface condition of concrete and brickwork could be modified further so that the bonding strength between coating material and concrete/brickwork could be improved.

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APPENDIX

A-1 Polypropylene Pipes for Storm and Sewer Water Applications

1. Introduction

Polypropylene(PP) also known as polypropene was initially produced commercially about 60 years ago after the successful development of a suitable catalyst, which enabled the polymer to have the type of structural characteristics useful for rigid items. Polypropylene (C_3H_6) belongs to a group of material called polyolefins. Composites based on these resins are relatively new as compared to other pipe materials (Karian, 1999). Increased interest in using this type of polymer is because of the remelt rheology and thermal behavior coupled with cost-effectiveness in a wide spectrum of applications (Karian, 1999). The density of the PP is approximately 0.9 g/cm^3 (0.0325 lb/in^3), considered to be the lowest among other plastic materials (Table A-1). Based on the thermal stability, low density, chemical and environmental inertness and simplicity of recycling PP is becoming an attractive alternative material for not only pipes but also in various other applications (Karian, 1999).

Chemistry

(i) Basic Molecular Structure

The basic material used to produce the polyethylene (PE) pipe is ethylene. For the other two plastics, modified ethylene is used as the basic material. In the basic ethylene ($CH_2 = CH_2$), unsaturated carbon compound, one of the hydrogen (H) is replaced by Cl^- for PVC and by $(CH_3)^-$ for propylene. Hence PVC basic molecule has the highest molecular weight of 63 (CH_2=CHCl) as compared to ethylene of 28 and propylene of 42 (CH_2=CH-CH_3).

Polypropylene consists of molecular chains with repeating units of propylene (CH_2=CH-CH_3; molecular weight of 42) monomer generated in the reactor. Propylene is also synonyms with propane, methyl ethane, methyl ethylene, 1-proplene and C3 hydro carbon (American Chemistry Counsel, 2007). It is derived from three major sources today (Karian, 1999). United

States is one of the countries that widely uses polypropylene in various applications (Karian, 1999).

2. Objectives

The overall objective of this study was to summarize the information available in the literature on polypropylene pipes for storm and sanitary sewer applications. The specific objectives are as follows:

(1) Material characterization and testing standards: Compare the properties of PP to other plastic pipe materials. Also compare the testing standards.

(2) Specifications for polypropylene pipes: Specifications for the use of PP pipes in various public and private sector projects.

(3) Standard test methods for polypropylene pipes: Review the standard testing for PP pipes.

(4) Case studies on the performance of plastic pipes: Compare the performance of plastic pipes.

(5) Third Party Testing of Polypropylene pipes: Summary of testing performed on the PP pipes.

3. Material Characterization

(i) Material Properties:

Typical properties of the three plastic materials are summarized in Table A-1 (PVC: http://www.sd-w.com/civil/pipe_data.htm; PE: George Fischer Piping Systems, 2010; PP: George Fischer Piping Systems, 2010). Polypropylene (PP) is a thermoplastic polymer similar to other plastic pipe materials such as polyethylene (PE) and polyvinyl Chloride (PVC) (Table A-2).

Density:

The density of PVC is 1.4 g/cm^3 (87.3 pcf) and is the highest of the three plastic materials. PE and PP plastics are lighter than water and PVC is heavier than water (Table A-1).

PVC is about 55% heavier than PP. Of the three plastic materials, PVC has the highest density and the PP has the lowest.

<u>Tensile Strength:</u>

PVC material has the highest tensile strength of 7450 psi. PE and PP pipe strength were close and are of 3600 psi and 3625 psi.

<u>a) Specific Strength:</u>

By definition, specific strength is the strength divided by the density. The specific strength of PVC is 12,300 N-ft/N. Also the specific strength of PP pipe is 9300 N-ft/N which is second highest. However the specific strength of PE is 8,700 N-ft/N.

<u>Flexural Strength:</u>

Flexural strength of PE100 pipe is 150,000 psi. PE100 stands out when compared its flexural strength with other plastic pipes. However PVC pipe with the values of 14,450 psi.

<u>a) Specific Flexural Strength:</u>

The specific strength of PE100 is higher than PVC pipe because of its high flexural strength. The specific strength of PE100 and PVC pipes are 362,319 N-ft/N and 23,845 N-ft/N respectively.

<u>Ratio of Flexural Strength to Tensile Strength:</u>

Ratio of flexural strength to tensile strength of PE pipe is 41.7 and PVC pipe is 1.94. Hence the flexural strength to tensile strength ratio of PE pipe is 20 times higher than PVC pipe.

<u>Linear Thermal Expansion:</u>

PVC pipe has the least coefficient of thermal expansion. PP pipe has the thermal coefficient close to PVC pipe when compared with PE pipe. The stresses induced at the joints because of thermal variation will get reduced in PP pipe.

Table A-1: Typical properties of plastic pipe materials

Properties		Unit	PE100***	PVC**	Polypropylene Natural PP-R*
Density		lb/cu.in (pcf)	0.0345 (59.6)	0.0505(87.3)	0.0325(56.2)
Tensile Strength@73^{o}F properties	Strength	psi	3600	7450	3625
	Specific Strength	N-ft/N	8,700	12,300	9,300
Modulus of Elasticity@73^{o}F	Strength	psi	130,000	420,000	130,500
	Specific Strength	N-ft/N	314,010	693,069	334,615
Flexural Strength@73^{o}F	Strength	psi	150,000	14,450	-
	Specific Strength	N-ft/N	362,319	23,845	-
Flexural Strength/Tensile Strength			41.67	1.94	-
Linear Thermal Expansion		in/in/˚F	$1.1x10^{-4}$	$0.3x10^{-4}$	$0.5x10^{-4}$

PE100*** - George Fischer Piping Systems, 2010.

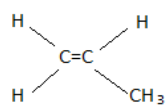
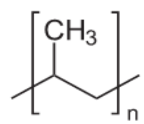
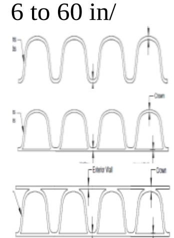
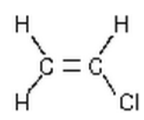
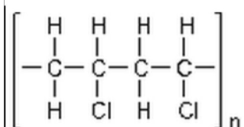
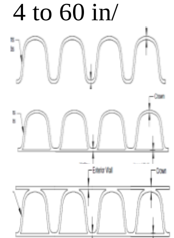
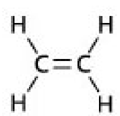
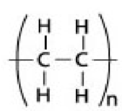
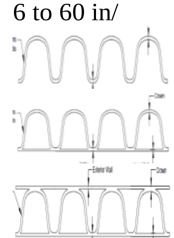
PVC** - http://www.sd-w.com/civil/pipe_data.htm.

Natural PP-R*- George Fischer Piping Systems,2010.

(ii) Testing Specifications

The Polypropylene (PP), PVC and Polyethylene (PE) pipes have similar types of pipe wall configurations – corrugated single, double and triple wall pipes (Table A-2). Pipe diameter of PP pipe and PE pipe varied from 6 to 60 inch. Unlike PP and PE, pipe diameter of PVC varied from 4 to 60 inch which was of a larger range (ASTM F 678, F 1803). PP has three ASTM standards whereas PVC and PE have about ten ASTM standards because PP is a recent addition to the list of plastic pipe. Also there are AASHTO, Canada and German standards for polypropylene pipes. PVC Pipes are covered by ASTM, AWWA and Canadian standards whereas PE pipes are covered by ASTM and AWWA standards. There are number of specifications to test and characterize the plastic pipes. PVC pipe alone has fourteen different ASTM test specifications (Table A-2). PE and PP have eight and three ASTM specifications respectively (Table A-2). There are also seven other ASTM specifications that can be used for all the plastic pipes. Also the CIGMAT joint infiltration test (http://cigmat.cive.uh.edu) is also a common test for the plastic pipes and other pipes. Hence for all the three pipes, there are about eight other standards (common) including CIGMAT standard. Both PVC and PE pipes have AWWA coverage. PP pipe has AASHTO specification (Table A-2).

Table A-2: Comparison of Material Characterization and Test Methods for Plastic

Type	Composition	Pipe Wall	Pipe Dimensions/ Cross sections (For Drainage Appl.)	ASTM Test Standards		Other Specifications	Remarks
				Special	Common		
Polypropylene 	Thermoplastic $(C_3H_6)_n$ 	Corrugated single wall, double wall and Triple wall.	6 to 60 in/ 	F2764, F2736, F2881	D 2321, F 1417-11a, F 1675-09, F 2422-05, D 2122-98, D 2412-11, D 2444, CIGMAT Joint Infiltration Test.	AASHTO MP21-11, CSA B182.13, DIN 8077, DIN8078	It exhibits excellent mechanical and chemical characteristics, including high strength, stiffness, resistance to stress crack propagation, & resistance to chemical (Hoppe ,2011).
PVC 	Thermoplastic C_2H_3 	Corrugated single wall, double wall and Triple wall.	4 to 60 in/ 	D 1784, D 1785, D 2152, D 2729, D 2774, D 3034, F 679, F 789, F 949, F 1336, F 1732, F 1760, F 794, F 1803		AWWA C605, C651, M 23, BNQ 3624-050, 3624-130, 3624-135	It is good in chemical resistance up to the temperature of 140^o F, corrosion, abrasion and wear resistance. It possess high resistance to fracture and impact damage (Uni-Bell PVC Association, 2001)
High density polyethylene 	Thermoplastic C_2H_4 	Corrugated single wall, double wall and Triple wall.	6 to 60 in/ 	F 2763, F 2762, F 2306, F 2947, F 2619, F 2487, F 2136, D 3350		AWWA C901, C 906.	PE piping retains its toughness even at lower temperatures. In addition, PE piping exhibits very high fatigue resistance. Also it resists the propagation of an initial small failure into a large crack (Plastic Pipe Ins.)

4. Testing Standards for Polypropylene

Both ASTM and AASHTO have standards dealing specifically with polypropylene plastic pipes for use in non-pressure applications (sanitary sewers, storm sewers, and drainage pipes). These testing standards were developed in years 2010 and 2011. The three ASTM standards were developed by committee F17.26 on Olefin based pipe and F17.62 on sewer pipes. ASTM F 2736 specifies the wall thickness of the single and double wall pipes. ASTM 2764 is for triple wall polypropylene pipe but no wall thickness is specified. AASHTO specification has a large variation in the wall thickness. The pipe stiffness requirement for AASHTO is almost double the ASTM requirement.

(i) Standards for PP pipe (Table A-3):

(a) Pipe size:

The range of pipe diameter covered varied from 12 to 60 inch in ASTM F 2881-11 and AASHTO MP 21-11. ASTM F 2764-10 includes only from 30 to 60 inch pipe diameter. ASTM F 2736-10 has a wider range of coverage from 6 to 30 inch in diameter pipes.

(b) Wall Thickness and Density:

The minimum wall thickness for PP pipe is in the range of 0.035 to 0.045 inch (ASTM F 2881-11, F 2736-10 and AASHTO MP 21-11). The maximum wall thickness of 0.108 inch is specified in ASTM F 2736-10 which is much higher than other ASTM and AASHTO standards. Also ASTM F 2764-10 has covered the wall thickness from 0.07 to 0.105 inch. The range of density for PP pipe is the same in all ASTM and AASHTO standards. The density varied from 0.0325 to 0.0345 lb/in^3.

(c) Melt Flow:

The value of melt flow rate of 0.25 to 1.5 g/10min at 230^{o}C is also the same in all ASTM and AASHTO standards.

(d) Strength and Elongation:

The ranges of tensile strength are the same in ASTM and AASHTO the tensile strength varied from 3500 to 4500 psi. The minimum and maximum elongation at yield of PP is 5 to 25% respectively for all ASTM and AASHTO standards.

(e) Flexural Modulus:

Flexural modulus varied from 175,000 to 325,000 psi in ASTM F 2881-11 and F 2736-10. However the value varied from 175,000 to 275,000 psi in ASTM F 2764-10 which has the same subcommittee (F 17.62) as ASTM F2881-11.

(f) Oxidation:

Oxidative induction time varied from 25 to 200 min in all ASTM and AASHTO standards.

Note: All ASTM and AASHTO standards recommended ASTM D 2412 for pipe stiffness test, ASTM D 2444 for impact test and ASTM D 3212 for joint tightness test.

Comparison of ASTM and AASHTO Standards:

Pipe Stiffness:

Pipe stiffness reduced from 70 to 25 psi with increase in pipe diameter in ASTM F 2881-11 and ASTM F 2736-10 has a minimum pipe stiffness of 46 psi at 5% deflection independent of the pipe diameter. For AASHTO MP 21-11, pipe stiffness varied from 19 to 65 psi. Though the ASTM subcommittee F 17.62 on sanitary sewer was the same for both ASTM F 2881-11 and F 2764-10, the pipe stiffness values required was different for the same pipe material.

Wall Thickness:

Also the minimum and maximum wall thickness required varied from 0.045 inch to 0.085 inch in ASTM F 2881-11, from 0.04 inch to 0.108 inch in ASTM F 2736-10, from 0.07 inch to 0.105 inch in ASTM F 2764-10. For AASHTO MP 21-11 wall thickness varied from 0.035 to 0.08 inch.

Table A-3: Selected standards for polypropylene (PP) pipe

ASTM std	Type of pipe	Pipe Size	Wall thickness	Density	Melt flow rate	Tensile strength	Elongation at yield	Flexural modulus	Min. pipe stiffness @ 5%	Remarks
F2881 - 11	Polypropylene (Corrugated single and double wall pipes)	12 to 60 inch	0.045 inch to 0.085 inch	0.0325-0.0343 lb/cu.inch	0.25 - 1.5 g/10 min at 230 deg C	3500-5000 psi	5-25 %	175000 - 325000 psi	25 - 70 psi *	Non-pressure storm sewer system
F2736 - 10	Polypropylene (Corrugated single and double wall pipes)	6 to 30 inch	0.04 inch to 0.108 inch	0.0325-0.0343 lb/cu.inch	0.25 - 1.5 g/10 min at 230 deg C	3500-5000 psi	5-25 %	175000 - 325000 psi	25 - 70 psi *	Non-pressure applications for sanitary sewers, storm sewers and drainage pipes.
F2764-10	Polypropylene (Triple wall pipes)	30 to 60 inch	0.07 inch to 0.105 inch	0.0325-0.0343 lb/cu.inch	0.25 - 1.5 g/10 min at 230 deg C	3500-4500 psi	5-25 %	175000 - 275000 psi	46 psi	Non-pressure sanitary sewer system
AASHTO MP 21-11	Polypropylene	12 to 60 inch	0.035 inch to 0.08 inch	0.0325-0.0343 lb/cu.inch	0.25 - 1.5 g/10 min at 230 deg C	3500-5000 psi	5-25 %	175000 - 325000 psi	19 psi - 65 psi *	Surface and subsurface drainage application
Remarks	Includes single,double and Triple wall pipes	Pipe diameter varied from 6 to 60 inches	Pipe wall thickness varied from 0.04 - 0.108 inches	Density is in the range of 0.0325-0.0343 lb/cu.inch	Melt flow rate is in the range of 0.25 - 1.5 g/10 min at 230 deg C	Tensile strength varied from 3500-5000 psi	Elongation at yield is in the range of 5-25%	Flexural modulus varied from 175000 - 325000 psi	Minimum pipe stiffness at 5% is in the range of 19 - 70 psi	Specifications for polypropylene pipes for storm sewer & sanitary sewer applications.

*** Pipe stiffness reduces with pipe diameter**

Standards for PE pipe (Table A-4):

ASTM standards cover the test methods for triple wall, annular, corrugated profile wall and fittings with the subcommittee of F17.62 on sewer. There are 8 ASTM standards on PE pipes only.

(i) Pipe Size:

Similar to PP pipe, the pipe diameter for PE pipe varied from 6 to 60 inch. The range of pipe diameter covered varied in each ASTM standard. ASTM F 2947-12 covers a wide range of pipe diameter from 6 to 60 inch. However in ASTM F 2763-11, the diameter considered varied from 30 to 60 inch. ASTM F 2762-11 and ASTM F 2306-11 have the pipe diameter range of 12 to 30 inch and 12 to 60 inch respectively.

(ii) Wall thickness:

The range of wall thickness for PE pipe is in the range of 0.035 to 0.079 inch in ASTM F 2306-11. In ASTM F 2763-11 and F 2762 -11, the wall thicknesses are in the range of 0.07 to 0.115 inch, 0.043 to 0.086 inch respectively. However ASTM F 2947-12 has covered wide range of wall thickness from 0.039 to 0.157 inch. Hence the wall thickness specified for PP pipes were thinner than the PE pipe.

(iii) Pipe Stiffness: The minimum pipe stiffness at 5% is in the range of 14 to 64 psi whereas for the PP pipe, the pipe stiffness varied from 19 to 70 psi according to AASHTO and ASTM standards. In ASTM F 2763-11 and F 2762-11, the minimum pipe stiffness is a constant value of 46 psi. However ASTM F 2306 and F 2947 have pipe stiffness varying with diameter.

Methods of Tests: As in PP pipe standards, all ASTM standards for PE pipe recommended ASTM D 2412 for pipe stiffness test, ASTM D 2444 for impact test and ASTM D 3212 for joint tightness test.

Applications: Like PP pipe, PE pipes are used for underground use for non-pressure sanitary sewer systems and gravity-flow storm sewer drainage purposes.

Table A-4: Selected standards for polyethylene (PE) pipe

ASTM std	Type of pipe	Pipe Size	Wall thickness	Min. pipe stiffness @ 5%	Pipe stiffness test	Impact test	Joint tightness	Remarks
F 2763- 11	Polyethylene (Corrugated triple profile wall pipes)	30 to 60 inch	0.07 inch to 0.115 inch	46 psi	According to ASTM D2412	According to ASTM D2444	According to ASTM D3212	Underground use for non-pressure sanitary sewer systems
F 2762 - 11	Polyethylene (annular and Corrugated profile wall pipes)	12 to 30 inch	0.043 inch to 0.086 inch	46 psi	According to ASTM D2412	According to ASTM D2444	According to ASTM D3212	Underground use for non-pressure sanitary sewer systems
F 2306/F 2306M-11	Polyethylene (annular and Corrugated profile wall pipes)	12 to 60 inch	0.035 inch to 0.079 inch	14 psi - 50 psi*	According to ASTM D2412	According to ASTM D2444	According to ASTM D3212	Underground use for gravity-flow storm sewer and subsurface drainage systems
F 2947-12	Polyethylene (annular and Corrugated profile wall pipes)	6 to 60 inch	0.039 inch to 0.157 inch	20 psi - 64 psi *	According to ASTM D2412	According to ASTM D2444	-	Underground use for non-pressure sanitary sewer systems
Remarks	Polyethylene (annular and Corrugated profile wall pipes)	Pipe diameter varied from 6 to 60 inches	Pipe wall thickness varied from 0.035 - 0.157 inches	Minimum pipe stiffness at 5% is in the range of 14 - 64 psi	Pipe stiffness test is according to ASTM D2412	Impact test is according to ASTM D2444	Joint tightness is according to ASTM D3212	General specifications for polyethylene pipes for storm sewer & sanitary sewer applications.

Standards for PVC pipe (Table A-5):

ASTM standards covered the test methods for PVC corrugated, large diameter sewer pipe and closed profile pipe and fittings with the subcommittee of F17.62 on sewer.

(i) Pipe Size:

Unlike other plastic pipes, PVC pipe has a wide range of sizes. The pipe diameter considered is varied from 4 to 60 inch. Also the range of pipe diameter covered varied in each ASTM standard. ASTM F 979-10 is covered the pipe diameter from 4 to 48 inch. However in ASTM F 679-08 and F 1803-06, the diameter considered varied from 18 to 48 inch and 18 to 60 inch respectively.

(ii) Wall Thickness: The range of pipe thickness of PP pipe covered only from 0.035 to 0.045 inch. However the wall thickness of PVC pipe is in the range of 0.022 to 1.822 inch. In ASTM F 679-08, three different ranges of wall thicknesses of 0.499 – 1.355 inch, 0.584-1.588 inch and 0.671-1.822 inch are mentioned. In ASTM F 949-10, two different types of wall thicknesses of 0.022 – 0.165 inch and 0.037-0.092 inch are mentioned. However ASTM F 1803-06 has covered small range of wall thickness from 0.07 to 0.24 inch.

(iii) Pipe Stiffness: The minimum pipe stiffness at 5% is in the range of 46 to 115 psi. The pipe stiffness requirement for PP pipe was 70 psi for a diameter of 12 inches. However the maximum pipe stiffness of PVC pipe is 115 psi which is 65% more than PP pipe maximum stiffness of 70 psi. Like above, in ASTM F 679-08, three different types of stiffness of 46, 75 and 115 psi are mentioned. In ASTM F 949-10, two different types of stiffness of 46 and 115 psi are mentioned. However ASTM F 1803-06 has specified the constant stiffness of 46 psi.

All ASTM standards recommended ASTM D 2412 for pipe stiffness test, ASTM D 2444 for impact test and ASTM D 3212 for joint tightness test.

(iv) Applications: PVC pipes are used for non-pressure drainage of sewage of surface water, sanitary sewer and storm sewers and perforated and unperforated pipes for subdrainage purposes.

Table A-5: Selected standards for poly vinyl chloride(PVC) pipes

ASTM std	Type of pipe	Pipe Size	Wall thickness	Min. pipe stiffness @ 5%	Pipe stiffness test	Impact test	Joint tightness	Remarks
F 679-08	Poly Vinyl Chloride (Large diameter pipes)	18 to 48 inch	0.499 inch to 1.355 inch	46 psi	According to ASTM D2412	According to ASTM D2444	According to ASTM D3212	Non pressure drainage of sewage of surface water.
		18 to 48 inch	0.584 inch to 1.588 inch	75 psi				
		18 to 48 inch	0.671 inch to 1.822 inch	115 psi				
F 949-10	Poly Vinyl Chloride (Corrugated pipe with a smooth interior)	4 to 48 inch	0.022 inch to 0.165 inch	46 psi	According to ASTM D2412	According to ASTM D2444	According to ASTM D3212	Underground use for non-pressure applications for sanitary sewers, storm sewers and perforated and unperforated pipes for subdrainage
		8 to 15 inch	0.037 inch to 0.092 inch	115 psi				
F 1803-06	Poly Vinyl Chloride (Closed profile pipe)	18 to 60 inch	0.07 inch to 0.24 inch	46 psi	According to ASTM D2412	According to ASTM D2444	According to ASTM D3212	
Remarks	Poly Vinyl Chloride (closed profile and Corrugated wall pipes)	Pipe diameter varied from 4 to 60 inches	Pipe wall thickness varied from 0.022 - 1.822 inches	Minimum pipe stiffness at 5% is in the range of 46 - 115 psi	Pipe stiffness test is according to ASTM D2412	Impact test is according to ASTM D2444	Joint tightness is according to ASTM D3212	General specifications for Poly Vinyl Chloride pipes for storm sewer & sanitary sewer applications.

5. Third Party Testing of Polypropylene

In Utah State University (Table A-6), parallel plate test was done with 30 inch diameter standard N12 type pipe. The pipe stiffness at deflection of 2.5% and 10% are 39.5 psi and 33.6 psi respectively. The peak force at 40% deflection is 165 psi, whereas for super-pipe type, the pipe stiffness at deflection of 2.5% and 10% are 69.4 and 60.2 psi respectively. The peak force at 31.1% deflection is 274 psi. From Fobbe Technical Group parallel plate test on N12 and twin wall pipe, the pipe stiffness varied from 58.9 to 111.9 psi at 5% deflection.

CIGMAT recently completed testing the double wall and triple-wall PP pipe joints of 30-inch diameter pipe. Based on the four joints tested, the following conclusions are advanced.

1. Straight Test:

There was no leakage at the 30-in. PP Pipe-joint (Triple Wall and Double Wall) when the joint was tested without any external loading for a total testing time of 30 minutes.

2. Shear Test:

The Triple Wall joint was subjected to a maximum shear force of 2870 lb. (equivalent to 96 lb/in diameter) and there was no leakage. The Double Wall joint was subjected to a maximum shear force of 2899 lb. (equivalent to 97 lb/in diameter). The total testing time was 2.5 hours for each test.

3. Angular Test:

During the angular test, the joint was subjected to a maximum rotation of 2^{o} at the joint. The total testing time was 2 hours and the shear load at the joint varied from 300 to 530 lb. during the angular test. No joint tightness test is specified in AASHTO. There was no leakage in all the tests.

Table A-6: Testing of Polypropylene pipe

			Pipe Stiffness (PSI)					Peak force		Leak
Third Party Study	Test type	Dia of pipe	Def (%):	2.50%	5%	7.50%	10%	% Def.	lb/in	NA
Utah State University	Parallel plate test	30"		39.5	38.8	36.4	33.6	40	165	NA
Utah State University	Parallel plate test	30"		69.4	69.6	65.5	60.2	31.1	274	NA
Fobbe Technical Group	Parallel plate test	12",15",18",24",30", 36" and 48"		-	58.89 -111.9	-	-	-	-	NA
CIGMAT (University of Houston)	Infiltration test	30"		NA	NA	NA	NA	2.97	96	zero

6. Conclusions

Based in the literature review and also testing of the polypropylene pipe joints the following conclusions are advanced.

1. Polypropylene pipes are relatively new plastic pipes in the market compared to PE and PVC pipes. The density and mechanical properties of PVC were higher than PE and PP. The density and mechanical properties of PE and PP are comparable, except the linear thermal expansion coefficient.
2. There are ASTM, AASHTO and German standards for testing and specifying the requirements for polypropylene pipes for sanitary and storm sewer applications. PVC pipe has the highest number of testing standards.
3. Number of cities and counties, have guidelines and/or specifications for using PP pipes in storm and sanitary sewer applications.
4. CIGMAT joint tests on 30-in diameter double wall and triple wall pipes have shown no leak at 7 psi infiltration pressure for joint rotation (2^{o}) and the shear load of about 3000 lbs at the joint.